A

TRANSACTIONS

OF THE

Association of Civil Engineers

CORNELL UNIVERSITY

1893

TRANSACTIONS

OF THE

ASSOCIATION OF CIVIL ENGINEERS

CORNELL UNIVERSITY.

VOLUME I. 1892-93.

CONTAINING

ADDRESSES BY NON-RESIDENT LECTURERS, MISCELLANEOUS PAPERS, CONSTITUTION, AND LIST OF MEMBERS OF THE ASSOCIATION.

NOTE.—This Association is not responsible for any statements or opinions advanced in any of its publications.

ITHACA, N. Y.

JUNE, 1893.

E. R. ANDREWS, Printer, 1 Aqueduct street, Rochester, N. Y.

CONSTITUTION

OF THE

ASSOCIATION OF CIVIL ENGINEERS OF CORNELL UNIVERSITY.

PREAMBLE.

We, the undersigned, members of the Senior and Junior classes in the college of Civil Engineering of Cornell University, do hereby form ourselves into an Association for the discussion of engineering topics, and the promotion of general information on engineering subjects, and do hereby agree to abide by, and sustain the following Constitution and By-Laws:

ARTICLE I.

NAME.

1. This Association shall be known as the Association of Civil Engineers of Cornell University.

ARTICLE II.

MEMBERSHIP.

1. The Association shall consist of Active and Honorary members.

2. All Alumni of this college and all students recognized as upperclassmen and registered in the college of Civil Engineering are eligible to membership in this Association.

3. Any eligible person may become an honorary member by a two-thirds vote of the members present at any regular meeting. Such members shall have privileges of active members except those of voting and holding office, and shall be exempt from all dues.

4. The membership fees of this Association for all active graduate members shall be *$3.00 per annum. All money received from membership fees shall be devoted to defraying cost of publication of non-resident lectures delivered before the Association. All other expenses of this Association shall be met by direct tax upon the undergraduate members.

5. A copy of each lecture delivered before this Association shall be forwarded to each member of the Association.

*This fee is to be changed, by amendment, to a more moderate sum for the coming year.

ARTICLE III.

OFFICERS.

1. The officers of the Association shall consist of a President, Vice-President, Recording Secretary, Corresponding Secretary and Treasurer.

2. The President shall preside at all meetings of the Association and enforce the Constitution and By-Laws, and shall call special meetings at the request of five active members.

3. The Vice-President shall take the chair at the request of the President and shall act as President in his absence. The Vice-President shall be Chairman of the appointment committee.

4. The Recording Secretary shall keep minutes of proceedings of all meetings of the Association and shall post notices for the same.

5. The Corresponding Secretary shall attend to all the necessary correspondence of the Association. He shall be elected from among the Faculty of the college.

6. The Treasurer shall receive all money and dues, and shall pay all bills of the Association, such bills to meet the approval of the Executive Committee before such payments. He shall make a report when called upon by the Association and also when his term of office expires. He shall be Chairman of the Executive Committee.

7. The officers shall be chosen by ballot at the last regular meeting of the spring term, from the Junior Class and shall hold office until their successors are elected.

ARTICLE IV.

COMMITTEES.

1. There shall be two Standing Committees, an Executive Committee and a Committee on Appointments. Each committee shall consist of three members and be appointed at the beginning of each term by the President.

2. The Executive Committee shall see that the rooms of the Association are ready for occupancy previous to all meetings and shall transact such business as may be referred to it by the Association.

3. The Committee on Appointments shall make appointments for all literary exercises for each meeting and such appointments shall be posted at least two days before reading. The committee shall furnish the Secretary with a list of such appointments.

ARTICLE V.

AMENDMENTS.

This Constitution or By-Laws may be amended by a two-thirds vote of all members present at any regular meeting; such amendment to be before the Association at least one week.

BY-LAWS.

ARTICLE I.

REGULAR MEETINGS.

Regular meetings shall be held on Friday of each week, in the Association rooms, commencing on the first Friday after registration week and ending on the last Friday but one before examination week of each term.

ARTICLE II.

QUORUM.

One-third of the active under graduate members of the Association shall constitute a quorum. No business can be transacted without a quorum being present.

ARTICLE III.

ORDER OF PROCEEDINGS AT A REGULAR MEETING.

1. Roll Call.
2. Minutes of Preceding meeting.
3. Literary Exercises.
4. Unfinished business.
    a. Report of Standing Committees.
    b. Report of Special Committees.
    c. Report of officers.
    d. Miscellaneous business.
5. New business.
6. Adjournment.

ARTICLE IV.

EXERCISES.

The exercises shall consist of discussions, memoirs, essays, papers, lectures, and such other exercises as the Association shall from time to time direct.

ARTICLE V.

SUSPENSION OF BY-LAWS.

A By-Law may be suspended for one meeting by a vote of two-thirds of the members present.

H. R. LORDLY,
E. J. FORT,
H. D. ALEXANDER.
Committee.

OFFICERS FOR 1892-93.

PRESIDENT,

HENRY R. LORDLY, *C. E. 93*.

VICE-PRESIDENT,

DAN. B. CLARK, *C. E. '93*.

CORRESPONDING SECRETARY,

PROF. CHAS. L. CRANDALL, *C. E. '72*.

RECORDING SECRETARY,

CHAS. W. ASHBY, *C. E. '93*.

TREASURER,

JOHN W. RIPLEY, *C. E. '93*.

COMMITTEES.

EXECUTIVE
- R. H. JACOBS,
- S. T. NEELY,
- JOHN W. RIPLEY, *Chairman*.

APPOINTMENTS
- C. G. ROSSMAN,
- I. W. BARBOUR,
- D. B. CLARK. *Chairman*.

PUBLICATIONS (Special)
- E. H. HOOKER,
- E. J. FORT,
- C. W. ASHBY.

PRESIDENT'S ANNUAL ADDRESS.

Gentlemen and Members of the Association of Civil Engineers, Cornell University:

Although there has been an Association of Civil Engineers in connection with our institution for twelve years now, it may be well and truly said that the Association of Civil Engineers, Cornell University, has just completed its first year.

You are all, no doubt, familiar with the difficulty of conducting an association of this nature on the former narrow limits of confining membership to upperclassmen alone, and it was with the fact in view that the new association was formed.

Under the present constitution, all graduates of the College of Civil Engineering, Seniors and Juniors, are eligible for membership according to their several classes. When all have signified their intention of joining, the membership list will amount to over four hundred.

The objects of the Association, primarily are as follows: (1) The furtherance of a professional intercourse among Cornell Civil Engineers, to their mutual advantage and to the wellfare of the College in general. This necessitates (2) the keeping of records of all graduates, who are kept informed of (3) the progress of our College by means of circular letters and the publications.

During the past year six lectures, by non-resident lecturers, have been delivered before the members in the institution. These lectures, besides results of laboratory experiments here, constitute the papers forming the first volume of the "Transactions." It is hoped that during the coming year, more material from graduate members will be at hand.

We have reason to be proud of the result of our efforts so far. The chief difficulty has been the locating of our graduates, but as the whereabouts of nearly all have been ascertained now, little trouble need be anticipated in the future, particularly if all notices of change of address are promptly sent in.

The weekly meetings held in College during the year were well attended; and many interesting and valuable subjects have been discussed. Such meetings will always prove beneficial to the Junior mem-

bers; and the papers from graduate members when published, cannot help but be valuable as an interchange of ideas.

As your President, the speaker begs to return thanks to the many courtesies extended to him during the year. The Association is especially indebted to Prof. C. L. Crandall, who so kindly consented to act as Permanent Secretary, to Mr. C. W. Ashby, Recording Secretary, and to Mr. John W. Ripley, Treasurer. The fulfilling of such offices means no light work and personal inconvenience to the holders, and these gentlemen deserve the heartiest thanks we can extend to them.

The committee on Publications, as well as those on Constitution, Appointments, and the Executive, have carried on their work in an efficient and business-like manner. The speaker also begs to extend to them the hearty thanks of the Association.

As regards finances, it is pleasing to know that, when a few more subscriptions are paid, the Treasurer will be able to meet all demands. When all the graduates extend their support, a more moderate fee may be exacted and the burden of support more evenly divided.

In the "Transactions," notices will be found of the death of several of our members. We regret greatly to chronicle such events, particularly sad in these cases on account of the youth of the deceased members and the promising careers which they had before them. The records of these men will always stand as valuable examples for our Junior members.

In conclusion the speaker thanks the Graduate Members for their encouragement and support. He trusts that the Association will see many successful and prosperous years and that it will stand as a bond of union among all Cornell Civil Engineers.

May 12th, 1893.

OFFICERS FOR 1893--94.

At the General Meeting May 12th, 1893, the following officers were elected for the ensuing year.

PRESIDENT,

HERBERT W. STRONG, '94.

VICE-PRESIDENT.

THOMAS S. CLARK, '94.

CORRESPONDING SECRETARY,

PROF. CHAS. L. CRANDALL.

RECORDING SECRETARY,

GEORGE F. BROWN, '94.

TREASURER,

JOHN W. TOWLE, '94.

An abstract from the address of retiring President will be found on another page.

ASSOCIATION OF CIVIL ENGINEERS.

CORNELL UNIVERSITY.

NOTICE TO MEMBERS.

A great many inquiries for assistants in engineering work are sent to the College, so many in fact that they have become an important aid in securing positions for the graduating classes. Frequently these inquiries are for men of experience in engineering work, or for professors for engineering schools.

By a little effort this system can be extended so as to aid not only the graduating class but other members who may be out of work or desirous of making a change, and those who may be in need of engineers or assistants.

If those desiring positions would notify the Secretary, stating experience, salary expected, etc., and those knowing of positions to be filled would also notify him, employers would oftener send here for engineers.

C. L. CRANDALL,

ITHACA, N. Y., June, 1893. *Secretary.*

MEMBERS OF

ASSOCIATION OF CIVIC ENGINEERS,

Cornell University.

ALL GRADUATES OF THE COLLEGE OF CIVIL ENGINEERING UP TO AND INCLUDING 1893.

* DECEASED. † NOT PRACTICING.

A.

†Allen, Chas. F. .... *C. E.* '73..German Nat. Bank, Denver, Col. in 1888.
†Alexander, Fred. B. .... *C. E.* '74........210 Macon Street, Brooklyn, N. Y.
Alexander, Henry D. .... *C. E.* '93...................... Minneapolis, Minn.
Ames, Willis C. .... *C. E.* '77....... 28 Calle de Ortezo, City of Mexico.
Locating Engineer.
Ashby, Chas. W. .... *C. E.* '93.............................. Troy, N. Y.
Atwood, Wm. G. .... *C. E.* '92........................... Fredonia, N. Y.
Aylen, Chas. P. .... *C. E.* '76................. Aylmer, Quebec, Canada.
Aylen, John C. .... *C. E.* '77................ Morrisburg, Ont., Canada.

B.

Bacon, Geo. M. .... *C. E.* '93..................... West Medford, Mass.
Baker, Chas. H. .... *C. E.* '86........... 51 Haller Bldg., Seattle, Wash.
Baker, Howard W. .... *C. E.* '86................... care of Chas. H. Baker.
Baldwin, Ernest H. .... *C. E.* '92.......................... Springfield, Mo.
Banks, John E. .... *C. E.* '92......................... Beaver Falls, Pa.
Draughtsman, Pittsburg Bridge Co.
Bardol, Frank B. E. .... *C. E.* '89........................... Buffalo, N. Y.
†Barros, Carlos .... *C. E.* '76......................... San Paulo, Brazil.
Battin, Henry W. .... *C. E.* '81............................ Winona, Minn.
Roadmaster, C. & N. W. R. R.
Beahan, Willard .... *C. E.* '76....... 240 Eleventh St., Jersey City, N. J.
†Bean, M. C. .... *C. E.* '72...................... McGrawville, N. Y.
Beardsley, Jas. W. .... *C. E.* '91.............................. Lemont, Ill.
Becker, Charton L. .... *C. E.* '88...................... Sterlingville, N. Y.
Beebe, Roscoe C. .... *C. E.* '92............................ Groton, N. Y.

Bellinger, Lyle F. .... *C. E.* '87 .... Atlanta, Ga.
R. R. Engineer.
Benson, Orville .... *C. E.* '88 .... Canton, Ohio.
Ass'st Engineer, Canton Bridge Co.
Beye, John C. .... *C. E.* '83 .... Lincoln, Neb.
Bishop, H. K. .... *C. E.* '93 .... Warsaw, N. Y.
†Bissell, Frank E. .... *C. E.* '78 .... South Bend, Ind.
Blake, Henry E. .... *C. E.* '73 .... 12 Walnut St., N. Adams, Mass.
Boright, Wm. P. .... *C. E.* '92 .... Chatham, N. Y.
Bowen, C. H. .... *C. E.* '93 .... Le Roy, N. Y.
Bowes, Thos. F. .... *C. E.* '91 .... Newton, Mass.
Bowman, Daniel W. .... *C. E.* '72 .... Phœnixville, Pa.
Boynton, Edw. P. .... *C. E.* '93 .... Cedar Rapids, Iowa.
†Bramhall, Wm. E. .... *C. E.* '77 .... St. Paul, Minn.
Brewer, J. C. .... *C. E.* '89 .... 319 Hudson Ave., Sandusky, Ohio.
Brown, Wm. .... *C. E.* '93 .... Belfast, N. Y.
Brownell, James P. .... *C. E.* '91 .... Carthage, N. Y.
Bruen, Frank .... *C. E.* '78 .... P. O. Box 253, New Haven, Conn.
Civil Engineer and Contractor.
*Bueno, F. de A. V. .... *C. E.* '76 ....
Burns, Justin J. H. .... *C. E.* '92 .... Watertown, N. Y.

C.

*Carpenter, Frank DeYeaux ..... *C. E.* '74 ....
Carpenter, Fred W. .... *C. E.* '84 .... Owego, N. Y.
Church, Irving, P. .... *C. E.* '73 .... Ithaca, N. Y.
Prof. Civil Engineering, Cornell University.
Clark, Dan. B. .... *C. E.* '93 .... Olean, N. Y.
Clark, Chas. H. .... *C. E.* '92 .. Wm. Whorton & Co., Philadelphia, Pa.
*Clark, Ira E. .... *C. E.* '72 ....
Clay, Francis W. .... *C. E.* '93 .... Richmond, Ky.
Colburn, D. Kent .... *C. E.* '72 .... Walnut Grove, Ill.
Collins, C. W. .... *C. E.* '89 .... Greenwich, N. Y.
Colnon, Redmond S. .... *C. E.* '87 .... Potsdam, N. Y.
†Conable, Morris R. .... *C. E.* '76 .... 82 E. 5th Street, St. Paul, Minn.
*Cook, Isaac N. .... *C. E.* '75 ....
*Cooper, Edgar H. .... *C. E.* '85 ....
Cornell, Oliver H. P. .... *C. E.* '74 ....
Couch, Vinton M. .... *C. E.* '92 ... Cor. Eleventh & Pike St., Pittsburg, Pa.
Crandall, Chas. L. .... *C. E.* '72 .... Ithaca, N. Y.
Prof. Civil Engineering, Cornell University.
Crane, Albert S. .... *C. E.* '91 .... Newton, Mass.
Crouch, Nelson S. .... *C. E.* '90 .... Erie, Pa.
Cummings, Elmore D. .... *C. E.* '89 .... Indiana, Pa.
Bridge Inspector.
Curtis, Chas. E. .... *C. E.* '85 .... Danby, N. Y.
Curtis, Grant .... *C. E.* '72 .... Pittsburgh, Pa.

Curtis, Winthrop L. ...... *C. E.* '92 .... Horseheads, N. Y.
*Curtis, Chas. W. .... *C. E.* '88 .... 925 F Street, Washington, D. C.

D.

Davis, Carl E. .... *C. E.* '91 .... Lemont, Ill.
Chicago Drainage Commission.
Davis, Chas. S. .... *C. E.* '89 .... Massillon, Ohio.
Chief Engineer Massillon Bridge Co.
Davenport, Ward P. .... *C. E.* '93 .... Plymouth, Pa.
Devin, George .... *C. E.* '73 .... Pottsville, Pa.
Chief Engineer Pottsville Bridge Co.
Dickinson, Joseph H. .... *C. E.* '90 .... Trenton, N. J.
Dingle, J. H. .... *C. E.* '92 .... Charleston, S. C.
Dimon, Henry G. .... *C. E.* '87 .... Groton, N. Y.
Ass't Engineer Groton Bridge Co.
Dillenbeck, Clark .... *C. E.* '88 .... Palatine Bridge, N. Y.
*Dodd, Franklin, M. G. ...... *C. E.* '90 ....
Dodgson, Frank L. .... *C. E.* '89 .... Batavia, N. Y.
Doerflinger, August .... *C. E.* '71 ...... 85 Lafayette Ave., Brooklyn, N. Y.
*Dobraluboff, J. H. .... *C. E.* '71 ....
Dole, Walter S. .... *C. E.* '92 ...... 125 Wisconsin Ave., Oak Park, Ill.
Doores, Wm. R. .... *C. E.* '93 .... Washington, D. C.
Dowling, Joseph L. .... *C. E.* '89 .... Brooklyn, N. Y.
Care National Transit Co., Lima, Ohio.
Duckham, Albert E. .... *C. E.* '89 .... Pittsburgh, Pa.
Duffies, Edw. J. .... *C. E.* '88 .... Markesan, Wis.
Dunn, Edw. Frank S. .... *C. E.* '92 .... Stewart Ave. & 40th St., Chicago, Ill.
Duryea, Edw. Jr. .... *C. E.* '83 .... Purdy's Station, N. Y.
Contracting Engineer.
Dyson, Jas. .... *C. E.* 78 .... Silverton, San Juan Co., Col.

E.

Eddy, Henry T. .... *C. E.* '70 .... Terra Haute, Ind.
Pres. Rose Poly. Inst.
Edwards, Jas. H. .... *C. E.* '88 .... East Berlin, Conn.
Ass't Engineer Berlin Bridge Co.
Emmons, Chas. M. .... *C. E.* '88 .... 121 Franklin St., Buffalo, N. Y.
U. S. Engineers' Office.
Ehle, Boyde .... *C. E.* '86 .... San Juan del Norte, C. A.
*Eidlitz, Allrad H. .... *C. E.* '76 ....
Eidlitz, Otto M. .... *C. E.* '81 .... New York City.
Engineer and Contractor, No. 128 E. 72nd Street.
†Erisman, Henry L. .... *C. E.* '92 .... Wilhelm, N. Y.
Etnyre, Sam'l L. .... *C. E.* '88 .... Oregon, Ill.
Ewing, Wm. B. .... *C. E.* '83 .... Chicago, Ill.
R. R. Engineer, 4136 Ellis Ave.

F.

Falkenan, Luis .... *C. E. '73* .... 53 Pine St., Chicago, Ill.
†Farmer, Wm. F. .... *C. E. '76* .... Nashua, N. H.
Farnham, Whitfield .... *C. E. '71* .... 6 Locust St., St. Louis, Mo.
Farnham, Irving F. .... *C. E. '92* .... Deposit, N. Y.
†Farrington, Wm. S. .... *C. E. '88* .... Syracuse, N. Y.
Engineering Employment Bureau.
Ferguson, Nicholas E. .... *C. E. '79* .... Stockholm, N. J.
Ferguson, Oscar W. .... *C. E. '75* .... 2128 McCausland Ave., St. Louis, Mo.
Ass't Engineer.
Ferris, Geo. F. .... *C. E. '81* .... Philadelphia, Pa.
Contractor.
Filkins, C. W. L. .... *C. E. '93* .... Olean, N. Y.
Fish, John C. L. .... *C. E. '92* .... Ithaca, N. Y.
Instructor, C. E., C. U.
Fisher, Bertrand H. .... *C. E. '85* .... Wellington, O.
*Fitch, Wm. R. .... *C. E. '74* ....
Fort, Edwin J. .... *C. E. '93* .... 3561 Rhodes Ave., Chicago, Ill.
Foster, Reuben B. .... *C. E. '74* .... South Lake Weir, Fla.
French, Jas. B. .... *C. E. '86* .... Richmond, Va.
Freeman, H. M. .... *C. E. '93* .... Orange Valley, N. J.
Frost, Fred W. .... *C. E. '72* .... Room 213 Stewart Bld'g, N. Y.
Frota, Antonio E de Maria ... *C. E. '77* .... Ceara, Brazil.
Fuertes, James H. .... *C. E. '83* .... Camden, Ark.

G.

Geigel, Antonio S. .... *C. E. '92* .... San Juan, Puerto Rico.
†George, Edward .... *C. E. '75* .... Nassau, Bahamas.
Gifford, Robt. L. .... *C. E. '91* ....
Gillette, Olin .... *C. E. '71* .... Medina, N. Y.
Golden, Harry E. .... *C. E. '91* .... Little Falls, N. Y.
Gordon, Fred F. .... *C. E. '93* .... Rochester, N. Y.
Green, Chas. N. .... *C. E. '88* .... Allentown, Pa.
(Assist. Supt. Consolidated Steel & Wire Co.)
Green, Robt. P. .... *C. E. '80* .... Media, Pa.
†Green, Wallace .... *C. E. '74* .... Clinton, Wis.
†Greene, Almon C. .... *C. E. '75* .... Palmyra, N. Y.
Greene, Carleton .... *C. E. '91* .... Altoona, Pa.
Greenawalt, Wm. C. .... *C. E. '87* .... 189 W. 136th St., N. Y. City
Architect.
*Gunner, Daniel W. .... *C. E. '87* ....
Guinn, John B. .... *C. E. '92* .... Georgia City, Mo.

H.

†Hadley, Eugene J. .... *C. E. '73* .... 6 Ashburton Place, Boston, Mass.
Halbert, Henry D. .... *C. E. '85* .... Vanceburg, Ky.

Hallock, Elijah A. .... *C. E.* '91 .... Massillon, Ohio.
Massillon Bridge Co.
Hart, Emmet E. .... *C. E.* '87 .... Little Valley, N. Y.
R. R. Eng.
Hasbrouck, Alvah D. .... *C. E.* '88 .... Highland, N. Y.
Hasbrouck, Chas. A. .... *C. E.* '84 .... Chicago.
Ass't Eng. American Bridge Works.
Haskell, Eugene E. .... *C. E.* '79 .... Washington, D. C.
Assist. U. S. Coast Survey.
Hatt, W. Kendrick .... *A. B. C. E.* '91 .... Cornell Univ., Ithaca, N. Y.
Instructor Civil Eng.
Havens, Rodman W. .... *C. E.* '80 .... Dallas, Tex.
City Eng.
Hawley, Abraham L. .... *C. E.* '86 .... Denver, Col.
R. R. Eng.
Hayes, Edw. .... *C. E.* '74 .... Cohoes, N. Y.
City Eng.
Hayford, John F. .... *C. E.* '89 .... Washington, D. C.
U. S. C. & G. Survey.
Hedden, Edw. .... *C. E.* '87 .... Ithaca, N. Y.
R. R. Eng.
Hedden, Elmond J. .... *C. E.* '92 .... Charlton, N. Y.
R. R. Engineer.
†Henderson, H. C. .... *C. E.* '72 .... Westchester, N. Y.
Herman, Robt. .... *C. E.* '79 ..... 455 I St., N. W., Washington, D. C.
Heustis, Chas. C. .... *C. E.* '92 .... Crown Point, N. Y.
†Hibbard, Horace M. .... *C. E.* '74 .... Ithaca, N. Y.
Hilburn, Edwin .... *C. E.* '91 ....
Hill, Theo. W. .... *C. E.* '93 .... Lyons, N. Y.
Himes, Albert J. .... *C. E.* '87 .... Oswego, N. Y.
Contractor.
Hilton, Edwin .... *C. E.* '91 .... 17 Seneca St., Hornellsville, N. Y.
Hitz, Irving .... *C. E.* '91 .... Longwood, Cook Co., Ill.
†Hoffeld, Henry R. .... *C. E.* '87 .... Lancaster, N. Y.
*Holbrook, E. M. .... *C. E.* '89 ....
Horner, Geo. W. .... *C. E.* '73 .... Belmont, N. Y.
†Howland, Rufus .... *C. E.* '72 .... Kingston, Pa.
*Hulse, Howard C. .... *C. E.* '91 ....
Hyde, Alfred T. .... *C. E.* '74 .... Oil City, Pa.
City Engineer.
Hyde, Edw. W. .... *C. E.* '73 .... Lincoln Ave., Walnut Hills,
Cincinnati, Ohio.

I.

Ingalls, Owen S. .... *C. E.* '86 .... Peterborough, N. Y.
R. R. Engineer.

J.

Jackson, Wm. G. .... *C. E.* '90 ....
Jacobs, Robt. H. .... *C. E.* '93 .... Ithaca, N. Y.
Janney, Wm. H. .... *C. E.* '74 .... Smyrna, Del.
Jarvis, Geo. M. .... *C. E.* '78 .... Denison, Texas.
Jordas, E. F. P. .... *C. E.* '74 .... San Paulo, Brazil.
Ass't Engineer, D. P. R. R.

K.

Kelley, Chas. L. .... *C. E.* '85 .... Arcadia, N. Y.
Kelley, Wm. D. Jr. .... *C. E.* '81 .... Kelley's Island, Ohio.
Kelsey, Clifford S. .... *C. E.* '88 .... Bridgeport, Conn.
Ass't Engineer.
Kelsey, Sidney .... *C. E.* '87 .... Kansas City, Mo.
Kennedy, Jas. C. .... *C. E.* '79 .... Glenwood Springs, Col.
Col. Midland R. R.
Knight, Frederick J. .... *C. E.* '73 .... Washington, D. C.
U. S. Geod. Survey.
Knighton, J. A. .... *C. E.* '91 .... Buffalo, N. Y.
Knoch, Julius J. .... *C. E.* '92 .... Saxonburg, Pa.
Krusi, Herman .... *C. E.* '82 ..... 42 Market Street, San Francisco, Cal.
San Francisco Bridge Co.

L.

Landon, Eugene A. .... *C. E.* '80 .... Groton, N. Y.
Chief Engineer Groton Bridge Co.
*Landers, Herbert H. .... *C. E.* '90 ....
Larned, Wm. H. .... *C. E.* '84 .... Poland, N. Y.
Lathrop, John P. P. .... *C. E.* '92 .... LeRoy, N. Y.
Lawrence, Theo. F. .... *C. E.* '88 .... Chester, N. Y.
Lawson, David T. .... *C. E.* '73 .... 56 Stone Street, New York City.
†Lay, C. H. .... *C. E.* '74 .... Oil City, Pa.
Lewis, Clarence C. .... *C. E.* '91 .... Eng. Dept., 210 E. Lexington St., Baltimore, Md.
†Lockwood, Ralph H. .... *C. E.* '73 .... Anthony, Harper Co., Kansas.
Lordly, Henry R. .... *C. E.* '93 .... St. John, N. B., Canada.
Lovell, Earl B. .... *C. E.* '91 .... Cortland, N. Y.
*Lyman, Geo. F. .... *C. E.* '73 ....

M.

MacHarg, John B. Jr. .... *C. E.* '93 .... Rome, N. Y.
Macpherson, David J. .... *C. E.* '77 ....
Makepeace, Mervale D. .... *C. E.* '75 .... Parmelia, N. Y.
Architect.
Mallery, C. S. .... *C. E.* '89 .... Groton, N. Y.
†Maltby, Albert B. .... *C. E.* '76 .... Indiana, Pa.
Mann, Louis M. .... *C. E.* '77 ....

†Marston, Anson............*C. E.* '89.......................... Ames, Iowa.
Prof. Civil Eng. Iowa Agric. Col.
†Marx, Chas. D.............*C. E.* '78.......................... Palo Alto, Cal.
Prof. Civil Eng., Leland Stanford, Jr. Univ.
Maxwell, Frank A..........*C. E.* '78 ....
McCormick, Cyrus H.......*C. E.* '78..........................Red Cliff, Cal.
Mining Engineer.
McCred, Clark W...........*C. E.* '81.......................... Leeper, Mo.
R. R. Supt.
*McMullen, J. C. ....*C. E.* '76 ....
Mead, Daniel W.............*C. E.* '84.......................... Rockford, Ill.
Contractor.
†Mead, Theo. L..............*C. E.* '77....................... Lake Charon, Fla.
†Meehan, John W. ....*C. E.* '87..........................Fairport, N. Y.
Menocal, Mario G...........*C. E.* '88.......................... Havana, Cuba.
†Merrill, Thos. D. ....*C. E.* '78..........................Saginaw, Mich.
Mersereau, Chas. V.........*C. E.* '79.......... 77 E. May Street, St. Louis, Mo.
Ass't Engineer Water Works.
Michaelson, Jos. McC.......*C. E.* '92.......................... Geneva, N. Y.
Moore, Frank C.............*C. E.* '92..............Franklyn St., Boston, Mass.
†Moraes, D. C. de...........*C. E.* '77....................... San Paulo, Brazil.
Moss, B. N. ....*C. E.* '93.......................... Ames, Iowa.
Mosscrop, Alfred M.........*C. E.* '85....................... Rochester, N. Y.
Ass't Engineer Rochester Bridge and Iron Works.
*Muller, Wm. ....*C. E.* '92 (died before graduating).
Münoz José del Carmen.....*C. E.* '91..................... Rivas, Nicaragua.
Murphy, Edw. C.............*C. E.* '84....................... Lawrence, Kan.
Prof. Civil Eng. Univ. of Kan.

N.

Nambu, Tsunejiro...........*C. E.* '88.......................... Tokio, Japan.
Niemeyer, Carl H............*C. E.* '91.......................Williamsport, Pa.
Northrop, Henry G. ....*C. E.* '74.......... 152 Gates Ave., Brooklyn, N. Y.
Norton, Geo. H...............*C. E.* '87.................... East Pembroke, N. Y.
Nye, Algernon S. Jr. ......*C. E.* '88 ......Croton Aqueduct Dept., New York.

O.

Ogden, H. Neely ....*C. E.* '89..........................Chicago, Ill.
Chicago Drainage Commission.
†Olin, Franklin W. ....*C. E.* '86..........................Troy, N. Y.
Onley, Willard ....*C. E.* '79....................Westernville, N. Y.
R. R. Engineer.
Ormsby, Frank W. ....*C. E.* '81.......................... Oswego, N. Y.
Ostrom, John N. ....*C. E.* '77............ 45 Broadway, New York City.
Bridge Engineer.

P.

Page, John...................*C. E.* '80..... .... Box 1207, Lima, Ohio.
Ass't Engineer United Pipe Line Co.

	Page, Wm. H.	*C. E.* '83	Care of Box 1207, Lima, Ohio.
†	Parke, R. H.	*C. E.* '79	
†	Parson, Frank	*C. E.* '73	
	Parsons, Herbert	*C. E.* '91	Bridgeport, Conn.
	Paz, Luis	*C. E.* '93	Santa Barbara, Honduras, C. A.
	Pearson, Edw. J. Engineer C. & N. W. R. R.	*C. E.* '83	Chicago, Ill.
	Perkins, Albert H.	*C. E.* '93	Ithaca, N. Y.
†	Perkins, P. H.	*C. E.* '75	Superior, Wis.
	Philips, Fred C.	*C. E.* '92	Fort Edward, N. Y.
	Pierce, Henry Engineer Maintenance of Way.	*C. E.* '80	Hinton, W. Va.
	Poss, Victor H.	*C. E.* '92	St. Louis, Mo.
	Potter, Fred H.	*C. E.* '93	Saginaw, Mich.
	Powell, Geo. W.	*C. E.* '85	Reed's Corners, N. Y.
	Preston, Erasmus D. Aid U. S. C. S.	*C. E.* '75	Washington, D. C.
	Preston, Edw. L.	*C. E.* '78	Newton, Iowa.
*	Preston, Kolce	*C. E.* '73	
†	Price, Chas. S.	*C. E.* '72	Johnstown, Pa.

R.

	Raymond, Chas. W. Supt. Iron Dollar Silver Mine.	*C. E.* '78	P. O. Box 583, Rico, Col.
	Read, Jesse E.	*C. E.* '81	Greenpoint, N. Y.
	Read, Willette W.	*C. E.* '88	Watertown, N. Y.
†	Reed, Jas. W.	*C. E.* '83	Caroline, N. Y.
	Ripley, John W.	*C. E.* '93	Sag Harbor, N. Y.
	Robinson, Horace B. Chief Engineer United Pipe Lines.	*C. E.* '74	Oil City, Pa.
	Rodriguez, Arturo Pittsburg Bridge Co.	*C. E.* '91	Beaver Falls, Pa.
	Rodriguez, Francis V.	*C. E.* '78	51 Villegas, Havana, Cuba.
	Roess, Gustav F. Assist. Engineer P. H. & A. R. R.	*C. E.* '90	Honesdale, Pa.
	Rogers, Alson	*C. E.* '72	Warren, Pa.
	Rogers, Jesse A.	*C. E.* '91	East Berlin, Conn.
	Rossman, Clark G.	*C. E.* '93	Ancram, N. Y.
†	Root, Francis J.	*C. E.* '73	Hamilton, N. Y.
	Rutledge, Arthur E.	*C. E.* '86	Rockford, Ill.

S.

†	Salmon, Sam'l W.	*C. E.* '71	Mt. Olive, N. J.
	Schoff, Frederic	*C. E.* '71	2220 Penna. Ave., Phila., Pa.
	Shillinger, John G.	*C. E.* '92	Denver, Col.
	Schualbach, Frank G. H.	*C. E.* '88	Menasha, Wis.

Shaler, Ira A.	*C. E.* '84	Purdy's Station, N. Y.
Contractor.		
Sheldon, Daniel C.	*C. E.* '83	Delphi, N. Y.
Shepard, Frank W.	*C. E.* '86	
Sherman, Walter J.	*C. E.* '77	Dallas, Tex.
†Sill, Cyrus B.	*C. E.* '72	740 Crossman Ave, Youngstown, Ohio.
Simpson, Geo. F.	*C. E.* '79	Lock Box 194, Harriman, Tenn.
Chief Engineer Harriman Coal & Iron Ry.		
Skinner, Frank W.	*C. E.* '79	277 Pearl St., New York
Editor Eng. Record.		
Skinner, John F.	*C. E.* '90	Rochester, N. Y.
Skinner, Joseph J.	*C. E.* '85	
Smith, Eugene R.	*C. E.* '77	Islip, N. Y.
*Smith, George La Tour	*C. E.* '71	
Smith, Leonard J.	*C. E.* '92	Cortland, N. Y.
Smith, Miller A.	*C. E.* '71	Brooklyn, N. Y.
Smith, William C.	*C. E.* '75	Bath, N. Y.
*Smith, William J.	*C. E.* '79	
Snider, Clarence A.	*C. E.* '91	1080 Wilson Ave., Cleveland, Ohio.
Steinacher, Gustavo J.	*C. E.* '92	Norwalk, Conn.
Sterling, Guy	*C. E.* '87	Gambier, Ohio.
Stewart, Clinton B.	*C. E.* '87	Wash. Univ., St. Louis, Mo.
Stewart, Neil, Jr.	*C. E.* '87	York, N. Y.
Stidham, Harrison T.	*C. E.* '91	1011 T St., N. W., Wash., D. C.
St. John, Richard C.	*C. E.* '87	277 Pearl St., N. Y.
Sanitary Engineer.		
†Stolp, Myron G.	*C. E.* '72	Aurora, Ill.
Stone, J. S.	*C. E.* '89	Am. Br. Works, Chicago.
Storey, Wm. R.	*C. E.* '81	Rochester, N. Y.
Stratton, Wm. H.	*C. E.* '88	East Berlin, Conn.
Assist. Engineer Berlin Bridge Co.		
Stubbs, Jas. H.	*C. E.* '76	Waltham, Mass.
Sugi, Bungo	*C. E.* '90	Japan
Sullivan, John	*C. E.* '88	Fisher's, N. Y.

T.

Tatnell, Geo.	*C. E.* '75	1421 Harrison St., Wilmington, Del.
†Thatcher, Cornelius S.	*C. E.* '78	St. Joseph, Mo.
Thomas, Howard	*C. E.* '77	West Superior, Wis.
Thomas, Seymour P.	*C. E.* '72	49 Williams St., N. Y.
Agent Phœnix Bridge Co.		
Thompson, Ellis D.	*C. E.* '76	Bound Brook, N. J.
Throop, Wm. B.	*C. E.* '77	Aurora, Ill.
†Tibbits, Addison S.	*C. E.* '77	Lincoln, Neb.
Tier, Lewis P.	*C. E.* '74	Norwalk, Ohio.
*Tilley, Geo. A.	*C. E.* '73	
†Tomlison, Frank C.	*C. E.* '74	Ironton, Ohio.
*Tompkins, John H.	*C. E.* '73	

Towle, Forrest M. .... *C. E.* '86 .... 26 Broadway, New York.
Eng. Dept. National Transit Co.
Trumball, Wm. C. .... *C. E.* '82 ....
†Turneaure, F. E. .... *C. E.* '89 .... Madison, Wis.
Prof. Univ. Wis.
Turner, Ebenezer T. .... *C. E.* '83 .... Ithaca, N. Y.
Turner, Horace G. .... *C. E.* '92 .... Watertown, N. Y.
Twining, Wm. .... *C. E.* '90 .... East Mauch Chunk, Pa.

U.

†Upjohn, Richard R. .... *C. E.* '80 .... Fond du Lac, Wis.

V.

Vedder, Herman K. .... *C. E.* '87 .... Lansing, Mich-
Prof. Engineering, Mich. Agr. Col.
Vedder, Wellington R. .... *C. E.* '91 .... Leeds, N. Y.
Vickers, Thos. McE. .... *C. E.* '91 .... Syracuse, N. Y.
Viegas, Muniz J. .... *C. E.* '77 ....
Vose, Walter I. .... *C. E.* '92 ....

W.

Wadsworth, Joel .... *C. E.* '90 .... Minneapolis, Minn.
Prof. Civil Engineering, University of Minn.
Wait, John C. .... *C. E.* '82 .... 111 Hammond St., Cambridge, Mass.
Instructor Civil Engineering, Scientific School.
Warner, Monroe .... *C. E.* '88 .... Pulaski, N. Y.
Warriner, Thos. R. .... *C. E.* '93 .... Adams, N. Y.
†Warthout, F. W. .... *C. E.* '74 .... Massillon, Ohio.
Washburn, Frank S. .... *C. E.* '83 .... Purdy Station, N. Y.
Contractor.
†Wasson, C. W. .... *C. E.* '74 .... Friendship, N. Y.
Webb, Walter L. .... *C. E.* '84 .... Philadelphia, Pa.
Prof. Civil Engineering, Univ. of Pa.
†Weed, Addison .... *C. E.* '79 .... New Hartford, N. Y.
Welker, Philip A. .... *C. E.* '78 .... Washington, D. C.
Aid U. S. Coast Survey.
†Wheelock, Chas. B. .... *C. E.* '76 .... 75 State St., Boston, Mass.
White, Timothy S. .... *C. E.* '73 .... Beaver Falls, Pa.
Vice-Pres. and Chief Engineer Penn. Br. Co.
Wick, Rich. B. .... *C. E.* '81 .... Verona, Pa.
R. R. Engineer.
Wightman, Willard H. .... *C. E.* '81 .... Ashland, Ore.
Wilcox, Robt. B. .... *C. L.* '90 ....
Williams, Chauncy G. .... *C. E.* '87 .... Cor. Eleventh & Pike St., Pittsburg, Pa.
Williams, Sylvester N. .... *C. E.* '72 .... Mt. Vernon, Ia.
Prof. Civil Engineering.
Wing, Fredk. K. .... *C. E.* '90 .... Buffalo, N. Y.
Wing, Chas. B. .... *C. E.* '86 .... Palo Alto, Cal.
Prof. Civil Engineering, Leland Stanford, Jr. University.

Memoirs of Deceased Members.

GEORGE LA TOUR SMITH, C. E. '74.

DIED JUNE 25, 1892.

GEORGE LA TOUR SMITH was born at Canandaigua, N. Y., September 3, 1847. He died at Harrisburg, Pa., from injuries sustained in the railroad collision near that city.

He was first educated at the Canandaigua Academy, and after graduating from that Institution attended the Yonkers Military Academy. The zeal and aptitude which he displayed for his work while pursuing the course in Civil Engineering at the Cornell University, and the high standing with which he graduated in 1874, gave an earnest of his future success. Having received his degree, he was appointed Superintendent of the U. S. Lighthouse Department, stationed at Baltimore, Md. After retaining this position for three years, he resigned to accept an appointment to the chair of Natural Sciences and Drawing at the Maryland State Normal School; which position he held until his death.

He was a contributor to several educational and scientific journals, and had long been engaged in the preparation for publication of a Manual of Laboratory work in Physics and Chemistry. Among the positions of honor which he held may be mentioned: Secretary of Photographic Society of Baltimore; Curator of the Maryland Academy of Science; and President of the Maryland Botany Club.

In 1878, he was married to Miss Lillie T. Armstrong of Baltimore, who, with one daughter, survives him.

He was a man of sterling character, untiring in his energies; and he was beloved and respected by all who came in contact with him.

ERNEST M. HOLBROOK, C. E. '89.

DIED OCTOBER 9, 1892.

HERBERT H. LANDERS, C. E. '90.

DIED FEBRUARY 4, 1893.

WILLIAM MULLER, '92.

DIED APRIL 28, 1892.

HOWARD CORWIN HULSE, '91.

DIED FEBRUARY 20, 1893.

There is a peculiar sadness in the death of a young man, who has just reached the field of action to the preparation for which he has devoted so much of his youthful energy, and postponed largely his enjoyment of so many of the common activities and interests of life. This painful sense of incompleteness, of the unfulfillment of a promising career, will be strong in those who learn of the deaths of those recent graduates, whose names precede this notice.

ERNEST MARTIN HOLBROOK, born at Kelley Islands, Ohio, May 29, 1867, was prepared for Cornell at the Oberlin Preparatory School. He graduated from Cornell in 1889 with the degree of C. E., and returned in the following year to conduct experiments in the Hydraulic Laboratory. In recognition of this work, he was granted the degree of M. C. E. in 1890. He then was employed at Lima, Ohio, in engineering work, on the completion of which he went in May, 1892, to Niagara Falls. While making a test of a motor October 9, he was struck on the head by a flying fragment from a pulley wheel which had burst. He died on October 14 from the injuries resulting from the blow.

He contributed to the Engineering Record an article embodying the results of his investigations in Hydraulics.

HERBERT H. LANDERS was born at Sidney, N. Y., November 16, 1868. Early in life he entered the Troy Academy, from which institution he came to Cornell to enter the course in Civil Engineering. By the time of his senior year, his ability and thoroughness had marked him out as one of the foremost of his class, and he was appointed Principal Assistant to the Chief Engineer in connection with the lake survey, and elected to the Society of the Sigma Xi.

On graduating, he accepted a position as draughtsman with Fort Worth and Denver Railway Company. He next entered the service of the New York Central, acting as Assistant Engineer in the survey of the Adirondack Railway, and later as Assistant Trainmaster on the Mohawk division of the main line. While superintending the breaking of a snow blockade near Utica in February last, he was struck and instantly killed by the Empire State Express, which had approached, unnoticed, by the track on which he had stepped to let pass a train just then relieved from the blockade.

It is with great sorrow that we chronicle the close of a career unusually full of promise.

WILLIAM MULLER, who was born in Warrenton, Virginia, January 12, 1866, came to Cornell from the Virginia Agricultural and Mechanical College, and entered with the class of '90, He interrupted his studies in order to spend two years in the office of the Keystone Bridge Co. and the Pittsburg Bridge Co. He then came back to the University, and in the spring term of his Senior year, April, '92, died from the results of an attack of Erysipelas.

HOWARD CORWIN HULSE was born August 12, 1868, and died at the home of his father, P. C. Hulse, in Brooklyn, on February 20, after a sickness resulting from an attack of pneumonia during the previous summer. He came from the Alexandria Military Institute to Cornell, and graduated in 1891. He was engaged on the survey for a sewage system in New England. His classmates will recall the gentleness of manner, and quiet geniality which made attractive an unusual determination of character.

STORM WATER IN TOWN SEWERAGE.

EMIL KUICHLING, M. Am. Soc. C. E.

One of the most important questions before the municipal engineer is that which relates to the disposal of the surface drainage of populous districts. The term "surface drainage" is commonly applied to the water which falls during rain storms, not only upon the inhabited area under consideration, but also upon the adjacent higher districts in the same valley. As will be seen in the following, the volume of such drainage water is frequently of enormous magnitude, and where adequate natural water-courses for its reception are not at hand, artificial ones must be formed by the engineer in order to prevent damage to public and private property by floods.

The public health is also directly affected by the degree of saturation of the soil and the extent to which putrescible organic matter is permitted to be washed from elevated surfaces and concentrated in depressions. A damp soil is generally believed to predispose to rheumatism, neuralgia and pulmonary troubles; and where the moisture is furthermore associated with much decaying organic matter, in or upon the ground, paroxysmal and malarial fevers are often induced. Numerous instances of outbreaks of malarial disease following excessive rainfalls and the saturation of the soil of populous districts, are recorded in medical and hygienic literature; and close observation has demonstrated that a marked improvement in the health of a community always results from drainage operations by which the surface waters are quickly collected and removed, and the ground rendered permanently drier.

Further argument as to the desirability of promptly getting rid of storm water in cities and towns seems to be unnecessary, since the annoyances resulting from its presence appear to have been recognized by all nations whose state of civilization induced families to aggregate into stationary communities. Recent explorations in Syria and Egypt have revealed that more or less elaborate provisions for the drainage of mon-

umental structures were made in the time of the Pharaohs; and the ruins of the various ancient civilizations of southern Europe give proof that the removal of the rainfall from populous localities received due consideration from both architects and rulers. The mere mention of the famous Cloaca Maxima and the Claudian Emissary will suffice to recall to mind how much attention was bestowed upon the subject of drainage by the Romans, and no demonstration is required to show that these structures have served as models for the modern drainage works of the largest cities of Christendom.

While a community remains small, a few narrow ditches discharging into the natural water-courses of the locality are usually sufficient to carry off the storm waters; but with its growth, the surface is rendered more and more impervious by roofs and pavements, until finally the rainfall accumulates to such extent as to cause these ditches and water-courses to become surcharged, whereupon an enlargement of their channels will be demanded. If the adjacent territory is valuable, such enlargement will generally result in the construction of covered drains or sewers, and to secure a further extension of this convenience in getting rid of the rainfall, similar branches will be laid to other portions of the occupied territory, until a more or less extensive net-work of underground conduits has been established. After a time as the development of the inhabited area proceeds, these conduits will usually become inadequate, and their reconstruction on larger dimensions will be required. In many cases this process is repeated several times, until the drainage problem finally receives proper study, and the sizes of the conduits are proportioned to the removal of the greatest volume of storm water which can reasonably be anticipated to reach them.

It is not intended at the present time to enter upon a discussion of the general expediency of removing the surface drainage of populous districts by means of underground conduits, either separately or in combination with the liquid wastes produced by the inhabitants. Suffice it to say that the practice is a common one, and that circumstances may easily be imagined wherein no other course is tolerable. The problem is therefore of interest to all young engineers, since it cannot be predicted how soon a thorough knowledge of the subject may be required of them in their professional career; also because comparatively little has yet been done to place it upon a scientific basis. Let it be our purpose to examine briefly the fundamental principles which enter into the problem, and endeavor to establish a process whereby we may be enabled to compute with approximate certainty the dimensions of such conduits.

The volume of water to be dealt with depends upon the intensity and duration of the rain, and the character, extent and shape of the surface upon which it falls. Of the water precipitated from the clouds, a portion is lost by evaporation, either during or soon after its fall; another portion penetrates into the soil or the porous rock which constitutes the natural surface, where it is either retained until dissipated by subsequent evaporation, or else percolates slowly through the permeable material to reappear at some lower level in the form of springs; another portion is absorbed by vegetation and the roofs and walls of buildings, as well as by the paving materials of the streets and the dust thereon; another portion is retained in surface depressions, thus forming pools of more or less magnitude, and the remainder flows off over the surface until it is collected in natural or artificial channels whence it is discharged into suitable outfalls. The proportions of these different components of the total precipitation upon a given area are by no means constant, but vary with the amount of moisture in the air and soil immediately before the storm; with the duration and intensity of the rain; with the peculiar character of the relatively impervious and porous areas, and with the extent and configuration of these areas.

If, at the beginning of a storm, the air and ground are already saturated from previous rains, the evaporation and absorption will be small, while the surface discharge will be large. For a long continued rain of uniform intensity, the drainage volume will gradually increase until a maximum is reached, which may be greater than the flow produced by a short rain of somewhat greater intensity. This fact is well illustrated by gravel walks or roadways, which at first rapidly absorb the water, but soon become saturated and shed most of the further precipitation as effectively as a non-absorptive surface. The same is also true of nearly every other kind of surface which it is necessary to take into consideration, since but a few varieties are absolutely impervious and smooth. It is also evident that as the area increases and its surface becomes more level, the greater will be the time necessary for the water to reach the collecting channels, and the greater the opportunities for evaporation, absorption and retention in pools or depressions; and conversely, when the area is small with steep slopes, the larger will be the percentage of the rainfall that will run off.

Obviously the storm flow from a given district will become a maximum when all portions of its surface are contributing to the discharge to their fullest extent. This, however, requires the lapse of a certain amount of time, both for the purpose of producing partial saturation of the surface, and for collecting the water in suitable channels or con-

duits. If the rain is of short duration, it is possible that the storm will cease before the flow from the more distant portions of the territory will reach the point of observation or general collection of the waters at the lower end of the district, and in that case the maximum discharge will probably ensue from a rain of longer duration and smaller intensity. In large districts it generally happens that a rapid rise in the outlet sewer will commence only after the storm has begun to abate in intensity, since in our climate the duration of heavy down-pours of rain is comparatively short. From these general considerations it will be seen that before much progress can be made towards a rational solution of the problem in hand, a more definite knowledge of rainfall phenomena is essential.

Inasmuch as we are usually called upon to deal with districts of moderate extent, such as from 100 to 1000 acres, and with considerable velocities of flow in the collecting channels, such as from 3 to 6 feet per second, it follows that in thickly populated areas the time required for the concentration of the storm water into the outlet sewer is not very great. Thus, in the case of a rectangular area of 500 acres, having a length of about 5,400 feet, a width of 4,000 feet, and a general slope sufficient to afford a velocity of 5 feet per second in the storm sewers when running about half-full, the greatest length of conduit will not exceed 7,400 feet, which will be traversed by the water in about 25 minutes; hence in this case, we are called upon to consider rainfalls of not more than one-half hour in duration. If, however, the dimensions of the area were only one-half as great as the figures just named, and the velocity of flow in the sewer or drain remained the same, all portions of the area would begin to contribute to the flow of the outlet after a period of only 15 minutes, a short time being allowed for the partial saturation of the absorptive surface and the establishment of a flow therefrom.

From these instances, it is clear that a knowledge of the greatest intensities of the rainfall in short periods of time must first be acquired, and that daily or even hourly rates of precipitation are of little value, since the maximum intensity of a severe storm in this portion of the country rarely continues for a longer time than about 30 minutes. Exact observations of these phenomena require the use of the most perfect self-registering rain gauges; and as these instruments are costly, reliable records of this kind have hitherto been scarce, and limited to only a few localities where elaborately equipped weather observatories are permanently maintained. The lack of numerous careful measurements in the past renders it somewhat doubtful how high the maximum

rainfalls in different periods of time should be assumed, and how often such excessive rains occur; but in order to present an approximate view of the subject, the following data, which have been compiled from a variety of sources, and relate to self-recording gauges with the exception of the last series, are herewith submitted:

TABLE showing Frequency of Severe Rainstorms of different duration, as observed in various American Cities.

CITY.	PERIOD.	No. of Years.	For durations of less than 10 minutes.		For durat'ns of 10 to 20 min.		For durat'ns of 20 to 30 min.		For durat'ns of 30 to 60 min.	
			No. of times.	Intensity in Ins. per hour.	No. of times.	Intensity in Ins. per hour.	No. of times.	Intensity in Ins. per hour.	No. of times.	Intensity in Ins. per hour.
New York, N. Y.	1869–91	22	7	5" to 7.5"	1	6.9"	1	5.0"	---	------
			3	4" to 5"	---	-------	---	-------	---	------
			5	3" to 4"	3	3" to 4"	4	2" to 3"	14	1" to 2"
Boston, Mass.....	1879–91	12	1	8.4"	2	4" to 5"	---	-------	2	2" to 3"
			1	4" to 5"	4	3" to 4"	---	-------	6	1" to 2"
Washington, D.C.	1871–91	20	2	8" to 9.6"	5	3" to 4"	1	3" to 4"	2	2" to 3"
			3	4" to 5"	---	-------	4	2" to 3"	14	1" to 2"
			10	3" to 4"	---	-------	---	-------	---	-------
Philadelphia, Pa.	1884–91	7	2	5" to 6"	2	4" to 5"	4	1" to 2"	2	2" to 3"
			3	4" to 5"	3	3" to 4"	---	-------	8	1" to 2"
			5	3" to 4"	---	-------	---	-------	---	-------
Chicago, Ill.	1889–91	3	1	5" to 6"	---	-------	---	-------	1	2" to 3"
			2	4" to 5"	---	-------	---	-------	2	1" to 2"
			2	3" to 4"					---	-------
St. Louis, Mo. ...	1889–91	3	1	4" to 5"	2	3" to 4"	---	-------	1	1" to 2"
Summary of ordinary observations at Rochester, Buffalo, Hemlock Lake, Ithaca and Oswego, all in western N. Y. (various periods).			4	3" to 4"	---	-------	---	-------	---	-------
	1871–88	18	2	2" to 3"	3	2" to 3"	1	2" to 3"	2	2" to 3"
				---------	---	-------	4	1" to 2"	7	1" to 2"

No automatic rain-gauge records are available for western New York, and hence municipal engineers in this part of the State are compelled

to make use of the usual daily observations, which contain only occasional measurements of the intensity of the precipitation during short periods. From a compilation of such records extending over a period of from 10 to 18 years at Buffalo, Rochester, Hemlock Lake, Oswego and Ithaca, which was made a few years ago by the writer, it was found that an intensity of from 2″ to 3″ per hour, lasting 20 minutes or less occurred 5 times; from 20 to 30 minutes, once; from 30 to 60 minutes, twice; and that an intensity of from 1″ to 2″ per hour, lasting from 20 to 60 minutes, occurred 11 times. Few, if any, of these records relate to showers during the night, and as heavy precipitations are more apt to occur after sunset while the atmosphere is cool, than during the daytime, it is very probable that these numbers should be considerably increased.

It should also be remarked that in our country the severest rains always occur during the months which are free from frost and snow, so that the problem is fortunately divested of the complications which would result if it were necessary to take into account a surface rendered more or less impervious by extensive freezing, or an additional volume of drainage water due to rapidly melting accumulations of snow. While the latter conditions generally produce disastrous floods from large water-sheds under the influence of warm rains, the drainage areas of municipal sewerage districts are usually too small, and the rate of snow melting too slow, to develop discharges of the magnitude experienced in the warm season. To illustrate this point, it may be assumed that at a time when a warm rain, continuing uniformly for 24 hours and yielding a total of 3 inches of water, may be expected, the snow lying upon the whole area is not more than a foot deep; also that not more than this depth will melt at a uniform rate during the period mentioned. Considering that the snow may be somewhat compacted by previous warm weather, the said depth of one foot may be regarded as equivalent to $1\frac{1}{2}''$ of water, thus giving a total depth of $4\frac{1}{2}''$ of water running off from the surface uniformly during 24 hours, which is at the rate of only $\frac{3}{16}''$ in depth per hour, or 121 cubic feet per second per square mile of drainage area. Spring floods of such magnitude, however, are exceedingly rare in our climate; hence it is evident that the figures above selected are excessive. On the other hand, in the case of an urban district of 2 square miles area, the roofs and pavements may easily be of such extent and character as to cause $\frac{1}{3}$ of the rainfall to concentrate quickly in the sewers, and the general grade of the surface may be such as to require not more than 30 minutes for the water from all parts of the area to reach the outlet sewer at the foot of the district. But from

the rain-gauge records above given, a rainfall lasting 30 minutes may occasionally yield a depth of 1 inch of water; and if one-third of this amount reaches the outlet before the rain ends, the discharge will be at the rate of $\frac{2}{3}''$ in depth per hour, or 430 cubic feet per second per square mile. The foregoing statement as to leaving winter conditions out of consideration in districts of comparatively small extent, is also borne out by the practical experience of municipal engineers, since nearly all of the notable cases of sewer engorgement are reported as having occurred in the warm months.

With regard to the extent of territory that may be covered by a heavy shower at any instant of time, observations show exceedingly wide variations. The storm area in our latitude is commonly several hundred miles in diameter, and occasionally exceeds two thousand miles in diameter, but the rain area is usually very much less, especially in the case of sharp thunder-storms where sometimes only a few square miles of the earth's surface are covered by the rain-cloud. From numerous observations of both the passage and approach of such clouds, the writer estimates that for a heavy precipitation lasting about 15 minutes, the least area covered by the densest portion of the rain is from 4 to 6 square miles. For shorter durations the area may be correspondingly smaller; but in general, the clouds which furnish rainfalls that are of moment in taxing the capacity of sewers, are considerably larger in extent than any single drainage area within ordinary municipal limits.

To indicate how great an area may be covered by a single storm, the following data relating to the great storm in Connecticut, on October 3 and 4, 1869 are cited, as given by the late Jas. B. Francis, C. E., and Prof. W. Upton: The area on which 8 inches or more fell, had a length of 65 miles, and an average width of 28 miles; the area on which 9 inches or more fell was about 50 miles long by 21 miles wide; the area on which 10 or more inches fell, was about 34 miles long by 15 miles wide; and the area on which 11 inches or more fell, had a length of about 20 miles, with an average width of 9 miles. Another memorable storm which extended over a large portion of New England, commenced on February 10, 1886, and lasted continuously for four days, during which 8 inches or more fell on 750 square miles, from 6 to 7 inches on 1,500 square miles, and from 4 to 5 inches on 2,750 square miles. These areas were computed by platting the observed depths of rainfall at different places on the same day upon a map, and then drawing the contours showing the lines of equal depth, whence the required areas were readily determined.

It has been sufficiently pointed out in the foregoing how important

it is to ascertain the duration of rainfalls of great intensity, and for this purpose it is often necessary to consider a particular portion of a shower, as the rates of precipitation at different intervals of time may vary within wide limits. Thus, for the first 5 minutes the water may fall at the rate of $\frac{1}{2}$ inch per hour, after which it will suddenly increase to a rate of 3 inches per hour for 15 minutes, and then abruptly change to a rate of $\frac{1}{4}$ inch per hour for 10 minutes, the total yield in the 30 minutes being $\frac{5}{6}$ inch, and the average rate being $1\frac{2}{3}$ inches per hour. Should the maximum discharge of the sewer receiving the drainage water actually be measured, and an attempt then be made to deduce the ratio of this discharge to the rate at which the water has fallen on the surface, it is easy to see how a grave error may result unless the actual maximum rate of precipitation is known. For example, if from an urban area of 100 acres the shower just mentioned produced in the outlet sewer a maximum flow of 75 cubic feet per second, and the grades were such as to admit of a concentration from all parts of the area within 15 minutes, the 3-inch rate of down-pour would give a delivery of about 300 cubic feet per second upon the area, and hence a ratio of of sewer discharge to rainfall of $\frac{75}{300} = \frac{1}{4}$; whereas, if the average rate of $1\frac{2}{3}$ inches per hour were taken, the estimated delivery would be about 170 cubic feet per second, and the ratio of sewer discharge to rainfall would become $\frac{75}{170} = 0.44$. From lack of proper data errors of this kind have frequently been made by eminent writers on sewerage, and have been copied by others until the subject has become needlessly involved in profound confusion and mystery.

In places where accurate records of the rainfall have been kept for a long term of years, the probable maximum rates of precipitation for various periods of time may easily be found in the following manner. The data should first be classified according to intensity of down-pour and duration, after which all intensities of more than one-half inch per hour should be platted on a diagram as ordinates to the corresponding durations in minutes as abscissas. A multitude of points in the area bounded by the two axes of co-ordinates will then be obtained; and since the rains of relatively light intensity are far more numerous than the heavy ones, the lower portion of the diagram will be thickly dotted with such points while in the upper portion they will be more or less straggling. By drawing a line through the series of highest points, we will have the envelope of the entire number, and the ordinates of this envelope will represent the probable maximum intensities of the rainfall in the locality for the corresponding abscissas or periods of time. It is obvious, however, that this envelope should be a regular line or

curve, and by omitting a very few of the highest points, such a line or curve may readily be constructed, and its equation found by trial. Although this method of procedure is quite simple, yet it seems to have been applied only within a few years past, so that the number of cases on record is very limited. To show what such equations of rainfall intensity are like, the following are herewith submitted, it being understood that y denotes the maximum intensity of the rainfall in inches per hour, while t denotes the duration of this intensity in minutes:

1) .... $yt = 360$, or $y = \frac{360}{t}$, Established in 1885 by Prof. F. E. Nipher for St. Louis, Mo.

2) .... $\begin{cases} y = 3.78 - 0.051\,t, \text{ for } t < 60 \\ y = 0.99 - 0.002\,t, \text{ for } t \geqq 60 \end{cases}$ Established in 1888 by Prof. Chas. D. Marx and the writer for Rochester, N. Y.

3) .... $\begin{cases} y = \frac{360}{t+30} \text{ for rare rainfalls,} \\ y = \frac{105}{t+15} \text{ for ordinary maximum rainfalls.} \end{cases}$ Established in 1891 by Prof. A. N. Talbot, for the Atlantic and Central States.

The formula of Prof. Nipher is the equation of an equilateral hyperbola, and was obtained by platting the records of the heaviest rainfalls observed at St. Louis, Mo., during a period of 47 years. The second formula was deduced from the rainfall records at Rochester, Buffalo, Oswego, Hemlock Lake and Ithaca, all in western New York, during periods of from 10 to 18 years, and the equations are those of two straight lines, which intersect near the ordinate for 60 minutes. The third formula represents the attempt which has recently been made by Prof. Talbot, of the University of Illinois, to deduce from the published records of the Weather Bureau and other sources, a general summary of the frequency, intensity and duration of heavy rainstorms in the Atlantic and Central States of the Union. It should be distinctly remembered that in all of the formulas y represents the *rate* of precipitation in inches per hour, and not the actual depth of water fallen; if the latter is wanted, the rate must be multiplied by the time in hours or fractions of an hour. To compare the results given by these different formulas for some particular case, let us assume that the time to be considered is $t = 30$ minutes, whence there follows:

a)	from Eq. 1:	$y = 12$	inches per hour, or a total rainfall of	6 in.
b)	" " 2:	$y = 2.2$	" " " "	" 1.1 "
c)	" " 3:	$y = 6$	" " " "	" 3. "
	and	$y = 2.3$	" " " "	" 1.2 "

Owing to the insufficiency of the data, it is unfair to assert that any one of the foregoing expressions is better than the others, as there may

easily be topographical pecularities in some particular districts, such as proximity to mountains, extensive plains, great lakes or the sea, which will materially modify the rates of precipitation. The safest way, under existing circumstances, is manifestly to study the rainfall characteristics of each locality by itself, although Prof. Talbot is of the opinion that the maximum rate of precipitation, in the different portions of the United States referred to, is quite uniform. It is to be hoped that the recent introduction by the Weather Bureau of a large number of self-recording rain gauges in the principal cities of the Union will result in furnishing engineers with the necessary correct statistics of rainfall phenomena.

From actual gaugings of sewers it is known that, within the ordinary limit of municipal sewer districts, the period of flood flow is practically of the same length as the duration of the storm, the maximum discharge being reached as soon as all portions of the area begin to contribute to the flow of the outlet; and it is also known that the heaviest storms extend over an area larger than ordinary sewerage districts. Accordingly, when the intensity and duration of the rain falling on a given district are known, the next question is, how much of it flows off. Obviously, this quantity will depend in the first place upon the proportion of the surface which is impervious, and in second place upon the facility with which the water is collected and concentrated in the outlet sewer. From a smooth copper roof-surface of considerable pitch, the whole of the rain will run off as fast as it falls; but if the same surface were truly horizontal, the water would accumulate until it had acquired a sufficient depth to form the head necessary for overcoming friction and enabling it to flow over the edges as fast as it fell. On the other hand, if the roof were formed of absorptive tile, a large quantity of water would be retained in the pores of the material, irrespective of its slope. Similar conditions also apply to the other varieties of surface which are encountered in practice. Urban districts covered with nearly impervious roofs and pavements, and provided with pipes, gutters and drains, yield a high percentage of storm water, while flat rural areas, with woods, meadows, and open cultivated soil, give off only a small proportion of the rain. From the best available measurements of discharge, it is found that at least 90 per cent. of the rainfall reaches the sewers in the most thickly settled portions of large cities where the pavements are impervious, smooth and clean; and that this percentage gradually diminishes in proportion as inferior pavements, unpaved yards, open spaces, lawns, gardens and parks make their appearance. It must also be remembered that in the latter cases the percentage will rise in pro-

portion as the absorptive areas become saturated, either from previous lighter showers, or from the long duration of rain of uniform intensity.

To illustrate this latter remark, as well as to indicate how much water runs off from a rural district, it may be proper to refer to the case of a certain territory of 357 acres in the city of Rochester, N. Y., from which the drainage discharge was closely measured for a number of sharply defined storms. About one-half of this area had at the time a relatively dense population averaging about 35 per acre, and was well developed, while the remainder was thinly settled and contained much agricultural or vacant land. Nearly all of the existing streets were graded and sewered, but only a small part of the aggregate length was improved with macadam and stone sidewalks, the rest having common earthen roadways and plank sidewalks. The soil is generally a clayey loam, interspersed with some gravel. The surface itself is slightly undulating, the average slope of the sewered streets being 1 in 150, while the sewer grades range from 1 in 47 for some of the small pipe sewers, to 1 in 910 for the main sewer, which is formed of good rubble masonry with a flat segmental invert of brick. The total length of main and tributary sewers above the point of observation was 10.35 miles, and the computed time required for the passage of storm water through the longest continuous line of sewer above said point was 34 minutes. It was also estimated that about 10 minutes more should be allowed in order to permit the water from the most remote locality to find its way into the nearest sewer opening. To secure the maximum flow from the whole district, it was accordingly necessary for the rain to last about 44 minutes at a uniform intensity. From the discharge computations, it was found that when the rain lasted from 30 to 40 minutes, about 12 per cent. of the precipitation per second ran off during the continuance of the maximum flow; whereas, when the rain lasted from 50 to 60 minutes, about 16 per cent. ran off. The rains in question were sudden showers of different intensities, ranging from 0.5 to 1.55 inches per hour, but were nearly uniform in rate for the periods mentioned.

An example of the conditions prevailing in an average urban district is afforded by the following: The tributary drainage area of 129 acres is generally well developed, and has the form of an irregular strip 4,800 ft. long by about 1200 ft. average width, beginning in the central part of the city and extending northerly towards the suburbs. The average density of population is about 32 per acre, and the area contains many large business blocks along the principal thoroughfare, while the remainder of the territory is occupied chiefly by residences of medium

size and good quality, standing on moderately large lots. The soil is mainly a clayey loam, with some muck in the lower districts, and the surface slopes gently to the north for about two-thirds of the length, after which it becomes very flat. All of the streets are sewered and graded, and about one-third of the aggregate length of roadway is paved with asphalt, stone blocks, macadam and gravel, the macadam, however, predominating in extent; the remainder of the roadways are of common earth. The average grade of the streets is 1 in 130, and the sewer grades range from 1 in 50 for some of the pipe tributaries to 1 in 630 for the outlet sewer, which is of rubble masonry in fair condition with a flat bottom. The total length of main and tributary sewers above the point of observation was 4.37 miles, and the computed time required for the passage of storm water through the longest continuous line of sewer above said point was 18 minutes, with an allowance of about 8 minutes more for full concentration in the sewers. A rain should therefore last about 26 minutes in order to secure a contribution to the outlet sewer from all parts of the area. For rains of from 20 to 30 minutes duration the discharge was found to be about 25 per cent. of the precipitation per second, and about 31 per cent. when the storm continued at uniform intensity for 50 to 60 minutes. The same rains falling upon a still better urban district of 133 acres yielded respectively about 35 and 41 per cent. of storm water in the outlet sewer; and from a small central district of only 25 acres, a yield of 58 per cent. was obtained.

Although these gaugings exhibited some discordances, yet the data were sufficiently numerous to admit of equalization by using the averages of the best established results; and while the figures given above may be only approximate, in consequence of imperfect estimates of the maximum intensity and duration of the rains, and of unavoidable errors in observing the flood marks left on the gauges, they nevertheless point unmistakably to the following general conclusions:

First:—The percentage of the rainfall discharged from any given drainage area is nearly constant for rains of all considerable intensities and lasting equal periods of time. This circumstance can be attributed only to the fact that the amount of impervious surface on a definite drainage area was also practically constant during the time occupied by the experiments.

Second:—The said percentage varies directly with the degree of urban development of the district; or, in other words, with the amount of impervious surface thereon. This fact is clearly shown by the large percentages derived from the relatively best developed district, in contrast with the smaller percentages obtained from the relatively less

improved districts, and to the still smaller results yielded by the least improved district; and it also serves to account for the constancy of the percentage discharged from any particular district for rainfalls of the same duration.

Third:—The said percentage increases rapidly, and directly or uniformly, with the duration of the maximum intensity of the rainfall, until a period is reached which is equal to the time required for the concentration of the drainage waters from the entire tributary area at the point of observation; but if the rainfall continues at the same intensity for a longer period, the said percentage will continue to increase for the additional interval of time at a much smaller rate than previously. This circumstance is manifestly attributable to the fact that the permeable surface is gradually becoming saturated and is beginning to shed some of the water falling upon it; or, in other words, the proportion of impervious surface slowly increases with the duration of the rainfall.

Fourth:—The said percentage becomes larger when a moderate rain has immediately preceded a heavy shower, thereby partially saturating the permeable territory and correspondingly increasing the extent of impervious surface.

Since the proportion of relatively impervious surface on the drainage area appears to be the most important factor in establishing the percentage of rainfall discharge, some attention should be given thereto. For practical purposes, the various kinds of such surface found in populous districts may be classified as follows: (1) The different varieties of roofs, from which nearly all of the water runs off. (2) First-class sidewalks and pavements, such as asphalt and cut stone blocks with asphalted or cemented joints. (3) Second-class sidewalks and pavements, such as ordinary rough stone blocks with large open joints leading directly to an absorptive foundation of sand, gravel, broken stone, etc. (4) Third-class sidewalks and macadam or gravel pavements. (5) Common earthen side-walks, roadways and other similar surfaces which are, however, well kept and graded, and properly drained. From the best pavements somewhat less water runs off than from roofs, owing to the proportionally greater irregularities of surface, and to the retention of much water by dust or dirt, even where the paving material itself is virtually non-absorbent. In the other classes of pavement, still larger quantities of water are lost by percolation through the joints, and by retention in the depressions and ruts. For the same reasons, a large loss occurs in the case of the earthen surfaces, even though they may have become well compacted by traffic, but the proportion running off will vary much with the slope and facilities for drainage. From permeable and flat

agricultural land, on the other hand, very little surface drainage will be obtained during the course of short and heavy showers, so that all such areas may be neglected in estimating the probable maximum storm-water flow in urban districts of moderate size. Areas of this kind, however, must be carefully considered when the territory is extensive, as may happen in the case of a brook which flows through the town or city.

The amount and quality of impervious surface on an urban drainage district is manifestly dependent upon the number of its inhabitants, since common experience teaches us that public and private improvements are inevitable as soon as the density of population reaches certain limits. A tedious analysis of the conditions in cities like Buffalo, Rochester and Syracuse, where the apartment-house system is still of limited extent, and by far the greater number of families live in detached dwellings, was made by the writer some time ago, from which it was found that, in well-developed urban districts, the density of population ranged from 20 to 50 per acre, with an average of 32 per acre; that the average number of dwellings per acre was 5, standing upon 6 lots; and that from 20 to 27 per cent., or an average of 24 per cent. of the entire surface was occupied by public streets and alleys, of which in turn 43 per cent., or about one-tenth of the whole surface, was provided with some kind of pavement. On classifying the data with respect to density of population, and assuming that the roofs are wholly impervious; further, that first-class walks and pavements will yield 80 per cent. of the rainfall, while the remaining three classes above described will yield respectively 60, 40 and 20 per cent., on an average; and finally by reducing the various partially permeable areas to wholly impervious ones, the following results were found:

TABLE showing relation of Density of Population to amount of fully impervious surface per acre of Urban Territory.

Average number of persons per acre.	Percentage of fully impervious surface.			Total percentage of fully impervious surface per acre.
	Roofs.	Improved streets.	Unimproved streets and yards, etc.	
15	8.4	3.3	3.0	14.7
25	14.0	7.0	4.3	25.3
32	18 0	10.2	5.0	33.2
40	22.5	14.7	5.4	42.6
50	28.0	19.0	5.6	52.6

By platting the final percentages as ordinates with the corresponding densities of population as abscissas, a curve may be drawn which will express the relation between these two variables, and its equation may also be found by trial, if desired. In applying the relation to cases where much greater densities of population occur than are given in the foregoing table, it must, however, be remembered that the rate of increase of the deduced impervious surface diminishes until a limit of 80 or 90 per cent. is reached for a density of about 75. Beyond this density there can be no material increase of such surface, since then the whole available area becomes covered with pavements and buildings, and any additional population is accommodated either by crowding more persons into the houses or by adding to the number of stories. It is also proper to remark that the figures given above refer only to certain average urban conditions, and are therefore subject to such modification as may be appropriate under different circumstances. For example, in a rapidly growing city, the present rural suburbs may soon acquire a far denser population and yield a much larger quantity of storm water than a casual examination might indicate; while in an old village, on the other hand, where the population has remained the same for many years, no further development can reasonably be anticipated.

A comparison of the foregoing final percentages of impervious surface on the drainage area with the actually measured percentages of rainfall that reach the outlet sewers, shows a very close numerical similarity. Advantage of this circumstance might be taken in attempting to establish a general formula which would express the relation between the average density of population in the district and the probable maximum volume of storm water per second from each acre of surface; but as the extent and character of the district must also be taken into consideration, which in turn largely influences the time required for the concentration of the drainage in the outlet, and hence also the computed rate of precipitation, it will be found that the resulting expression will become too complicated for convenient practical use. It may, however, be of interest to indicate how this can be done, and the following notation will therefore be taken:

A = magnitude of the entire drainage area in acres;
Q = maximum sewer discharge from A in cubic feet per second;
y = maximum intensity of the rainfall in inches per hour;
t = duration of y in minutes;
b and c = constants in the relation between y and t;
m = proportion of impervious surface on A, which will also be assumed as equal to the proportion of rainfall discharged during the period of greatest flow.

Now since the gaugings indicate that the proportion of storm water varies directly with the duration t from the beginning of the rain up to the instant of maximum flow, we may in general place $m = at$, where a is some constant; and as a precipitation of one inch per hour is very nearly equal to one cubic foot of water per acre per second, we also have: $Q = myA$. For the relation between t and y we may take the simple expression already given, viz: $y = b - ct$. By substituting these values of m and y in the expression for Q, we have:

$$Q = Aat\ (b - ct),$$

which is an equation of the second degree with respect to the time t. It is probable, however, that the relation between m and t is not linear, as above considered, but quadratic, in which event the last expression for Q would become a cubic equation; and the complication would be still further increased if it were attempted to introduce some involved relation between m and the number of persons per acre. Furthermore, inasmuch as the values of the various factors are still invested with more or less uncertainty, it seems useless to continue this effort of formulation until better data are at hand.

The foregoing investigation is nevertheless of some interest, as it demonstrates that Q is a function of t^2 at least, and must therefore attain a maximum for some particular value of t. If we accept provisionally the above expression for Q, and differentiate it with respect to t as variable, the first differential coefficient will become:

$$\frac{dQ}{dt} = Aa\ (b - 2ct);$$

and by placing this coefficient equal to zero, we will obtain the condition under which Q will become a maximum, viz:

$$Aa\ (b - 2ct) = 0, \text{ whence } t = \frac{b}{2c};$$

but from the writer's examination of the rainfall statistics for western New York, it was found that $b = 3.73$ and $c = 0.051$; hence in this locality Q would become an absolute maximum for $t = 37$, or for a rainfall having an intensity of $y = 1.84$ inches per hour lasting uniformly for 37 minutes, provided that the drainage area were of such size and slope as to insure the concentration of the storm waters within this period of time. It should be distinctly remembered, however, that this conclusion is entirely contingent upon the accuracy of the preceding premises, and is by no means set forth as an actual fact.

Various attempts have heretofore been made to express in mathematical terms the relation between the storm-water discharge, the drainage

area, the rainfall and the general slope of the surface, but experience has shown that they have all been more or less unsatisfactory. The best known of these formulas are those of Hawksley, Bürkli-Ziegler, Adams and McMath,* and a brief reference to them may not be out of place. Hawksley endeavored to find the relation between the diameter of a circular conduit or sewer and the other factors above named, on the assumption of a rainfall of one inch per hour, of which one-half would reach the sewer, and of a sewer grade parallel with the surface. After many trials he finally invented the empirical formula which bears his name, and of which the others are modifications. Bürkli-Ziegler's formula is merely Hawksley's, expressed in a somewhat different shape and provided with a variable coefficient depending upon the intensity of the rainfall, thereby giving it a wider range of application. Col. J. W. Adams, in his treatise on sewerage, states that while Hawksley's formula "gives ample capacity for the smaller dimensions of sewers and for limited areas, it did not prove so satisfactory in the larger," and he accordingly proposes a different empirical expression which, while "giving slightly less results in the smaller areas, gives the increased dimensions in the larger that experience has pointed out as desirable in this locality" (Brooklyn, N. Y.). McMath's formula is modeled after Bürkli-Ziegler's, but has a different empirical exponent, so that materially different results are obtained.

For the purpose of more conveniently comparing these four empirical formulas with each other, as well as with reliable observations, the writer has reduced them all to the same notation; and to make the first and third applicable to other rates of rainfall than one inch per hour, this factor has been introduced in making the necessary transformations. Placing $Q =$ maximum discharge of the outlet sewer in cubic feet per second; $y =$ maximum rate of rainfall in inches per hour, which is practically the same as if expressed in cubic feet per acre per second; $A =$ magnitude of the drainage area in acres; and $s =$ sine of the general slope of the surface, or the quotient of the average fall divided by the average length, we will have:

1). Hawksley .... $Q = 3.946\ Ay \sqrt[4]{\frac{s}{Ay}}$.

2). Bürkli-Ziegler .... $Q = C.\ Ay \sqrt[4]{\frac{s}{A}}$, where the value of the coefficient C varies from 1.757 to 4.218, with an average of $C = 3.515$.

3). Adams .... $Q = 1.035\ Ay \sqrt[12]{\frac{s}{A^2 y^2}}$.

* See Appendix.

4). McMath .... $Q = C.\ Ay \sqrt[5]{\frac{s}{A}}$, where the value of the coefficient C varies from 1.284 to 2.986, with an average of $C = 2.488$.

It should be remarked that the first and third expressions relate to ordinary urban conditions of surface, and are designed to apply best when $y = 1$; while in the second and fourth expressions, the smaller coefficients refer to suburban, and the larger to densely populated districts, the average values referring to the same conditions assumed in the first and third. It will be observed that in formulas 1) and 3) the ratio $\frac{Q}{Ay}$ will diminish as the intensity y of the rainfall increases; but since the fundamental principles of hydraulics teach that the resistances to flow diminish rapidly with an increase of depth or volume, it is evident that there is a defect in these expressions which will manifest itself conspicuously in the case of small areas. For large districts, on the other hand, it may be conceded that the said ratios may not increase perceptibly within the range of usual intensities; nevertheless there is certainly no reason apparent why they should diminish when the rate of rainfall increases. The only justification for such a diminution lies in the circumstance that very heavy intensities usually last only a short time, and that consequently the whole area may not be contributing to the observed maximum discharge; but as this depends entirely upon the form, magnitude and slope of the territory, it is obvious that the said first and third formulas must be used cautiously. The second and fourth expressions are free from the foregoing objections, but introduce an element of uncertainty as to the choice of the proper coefficient, which at once brings us back to the exercise of mature judgment.

A safer method of computing the probable maximum storm-water discharge is undoubtedly the one which has herein been described. By its adoption, the engineer is compelled to make a thorough study of the problem before him, and to take counsel with intelligent citizens as to the future development of the various districts which are to be drained. Each step in the process is sharply defined and susceptible of amendment, if necessary, and is moreover founded upon a better array of ascertained facts than is the case with any of the empirical formulas mentioned. In applying it, the probable future amount of impervious surface on the given areas must first be estimated, either with reference to the density of population or in any other more reliable manner that may be devised; it may then be assumed that all of the water which falls upon such surface will run off without loss; further, since the topography of each area must in any case be known, the lengths of the

longest tributaries to the outlet sewer can readily be determined, as well as their approximate diameters, and thence also the velocities of flow therein; from these elements, the time required for the flood-waters to reach the outlet sewer from the most distant points in the area can next be found; and when an examination of the local rainfall statistics has revealed the proper relation between the probable maximum intensity of the precipitation and its corresponding duration, the maximum rate of rainfall belonging to the time so found can then be deduced. With the notation previously adopted, we will then have for each area the discharge:

$$Q = my\,A, \qquad (5)$$

from which the proper dimensions of the channel or conduit may easily be computed or taken from suitable tables.

When the drainage area is very large and flat, so that the time t required for the concentration of the storm-waters from all portions of the territory in the outlet sewer becomes longer than about 40 minutes, it may happen that the computed value of Q for the entire area will be less than that which results from the use of only a portion thereof, with a correspondingly smaller value of t and larger value of y. In such cases the absolute maximum value of Q must be found by trial, which is quickly done, inasmuch as every large area must necessarily be subdivided into a number of smaller districts, each of which is treated in the manner above described. By omitting successively some of the upper subdivisions and computing new values of y from the resulting smaller values of t, or shorter lengths of tributary sewers, a series of new values of Q will be found, of which the largest must be selected for fixing the dimensions of the channel.

In conclusion, it may be of interest to make an application of the above method of computing the storm water discharge, and compare the result with the figures given by the other four formulas. For this purpose let us take a district of 360 acres, which may in the future be constituted as follows: 60 acres of first-class territory containing 55 per cent. of impervious surface, and a population of 3,000; 90 acres of second-class territory containing 45 per cent. of impervious surface, and a population of 3,600; 120 acres of third-class territory containing 33 per cent. of impervious surface, and a population of 3,600; and 90 acres of fourth-class territory, containing 20 per cent. of impervious surface and a population of 1,800. On the whole area there is accordingly 131.1 acres or 36.4 per cent. of impervious surface, and a population of 12,000, thus giving an average density of 33⅓ persons per acre. These conditions will at once be recognized as fairly representing a medium

urban territory, to which any of the four formulas are directly applicable. Furthermore, let it be assumed that the time required for the concentration of the storm-waters at the lower end of the district is $t = 40$ minutes, and that the average surface slope of the streets is $s = \frac{1}{150}$, while the grade of the outlet sewer is 1 in 500. From the relation between the probable maximum intensity of the rainfall y and the time t for this locality, we have: $y = 3.73 - 0.051\, t = 3.73 - 2.04 = 1.69$ inches per hour, or cubic feet per acre per second; and as this is assumed to fall upon and run off without loss from 131.1 acres of impervious surface, the volume from this source will be: $Q_1 = 221.6$ cft. per second. But as the rain is assumed to last uniformly for so long a time as 40 minutes, the remaining permeable or absorptive area may be regarded as having become partially saturated, and as contributing towards the close of the rain at least 10 per cent. of the precipitation thereon; hence we will have the additional volume from the 228.9 acres of permeable surface: $Q_2 = 228.9 \times 0.169 = 37.7$ cft. per second. The total storm-water discharge will thus amount to:

$$1). \quad Q = Q_1 + Q_2 = 259.3 \text{ cft., say 260 cft. per second.}$$

On the other hand, we will obtain from Hawksley's original formula as transcribed, which contemplates that $y = 1.0$ and that s is the sine of the slope of the outlet sewer, or in this case $s = \frac{1}{500}$:

$$2). \quad Q = 3.946\, A \sqrt[4]{\frac{s}{A}} = 68.97 \text{ cft., say 69 cft. per second;}$$

or, if the formula be equipped with a factor representing the rainfall, and s taken as referring to the average surface slope instead of to the grade of the outlet, we must introduce the values $y = 1.69$ and $s = \frac{1}{150}$, whence:

$$3). \quad Q = 3.946\, Ay \sqrt[4]{\frac{s}{Ay}} = 138.13 \text{ cft., say 138 cft. per second.}$$

From Bürkli-Ziegler's formula as above transcribed, with the average value of the coefficient $C = 3.515$, $y = 1.69$ and $s = \frac{1}{150}$, we find:

$$4). \quad Q = 3.515\, Ay \sqrt[4]{\frac{s}{A}} = 140.29 \text{ cft., say 140 cft. per second.}$$

The difficulty here is to determine what value shall be given to y, since Bürkli-Ziegler distinctly states that it should be the maximum which obtains during the continuance of the storm, and he assigns to it values ranging from 1.75 to 2.74 for central Europe. If an irregular rain lasting 40 minutes be assumed, it is easy to see that a maximum intensity of 3 inches per hour might prevail for at least one-half the

time with a much smaller rate for the remainder, but yet giving the same average of 1.69 inches, as taken above; and with $y = 3.0$ instead of 1.69, we will have: $Q = 249.0$ cft. per second, or very nearly the same as the quantity computed by the writer's method.

From Adams' original formula as transcribed, which contemplates that $y = 1.0$ and that s is the sine of the slope of the outlet sewer, or in this case, $s = \frac{1}{500}$, we have:

$$5). \quad Q = 1.035\ A \sqrt[12]{\frac{s}{A^2}} = 83.23 \text{ cft., say 83 cft. per second;}$$

whereas if we use the formula as given in the foregoing, with the factor y properly introduced and the substitution of the general surface slope $s = \frac{1}{150}$ in place of the sewer grade of $\frac{1}{500}$, we obtain:

$$6). \quad Q = 1.035\ Ay \sqrt[12]{\frac{s}{A^2 y^2}} = 142.48 \text{ cft., say 143 cft. per second.}$$

From McMath's formula as transcribed above, we will find with $y = 1.69$, $s = \frac{1}{150}$, and the average value of the coefficient $c = 2.488$:

$$7). \quad Q = 2.488\ Ay \sqrt[5]{\frac{s}{A}} = 171.23 \text{ cft., say 171 cft. per second;}$$

but if y were taken at the value adopted by Mr. McMath for the city of St. Louis, Mo., viz: $y = 2.75$ inches per hour, we would have: $Q = 278.0$ cft. per second, or somewhat more than the discharge as computed by the writer's method.

It has thus been shown how the volume of storm-water from a populous district may be calculated, and the variations of a number of different methods of computation have likewise been indicated. In selecting such a method, it will be wise to bear in mind that with the heavy rains of frequent occurrence in this country, the proportioning of sewers by Hawksley's formula has in many cases resulted in floodings, and that an extensive experience with the other methods or formulas has not yet been gained. On all sides, however, it is being conceded that much larger quantities of storm-water run off from urban surfaces than was formerly supposed, and hence it is obvious that a thoroughly rational method of sewer computation is urgently demanded. An attempt has been made in the foregoing to establish such a process; but as much room for improvement in this direction is still left, it is hoped similar investigations by other engineers will soon provide us with an array of facts sufficient in number to remove the subject wholly from the domain of doubt.

APPENDIX.

Hawksley's formula appears to have been established at some time between the years 1853 and 1856, and to express analytically the relation between the diameter and slope of a circular outlet sewer and the magnitude of its drainage area, which is embodied in a table prepared in 1852 by John Roe, Surveyor of the Holborn and Finsbury sewers (London), after numerous observations of their storm discharge. As rains yielding more than one inch in depth per hour are of comparatively rare occurrence in London, an intensity of one inch per hour was then probably regarded as a maximum for which provision should be made in municipal sewerage work, and the diameters, grades and areas given by Mr. Roe were considered as applicable to such intensity. In its original form, Hawksley's formula is (See Report of Commission on Metropolitan Drainage, London, 1857):

$$log.\ d = \frac{3 log.\ A + log.\ N + 6.8}{10},$$

where: d = diameter in inches of a circular sewer adapted to carry off the storm water due to a rainfall of one inch per hour;

A = magnitude of the drainage area in acres;

N = length in feet in which the sewer falls one foot.

If we replace N by its equivalent $\frac{1}{s}$, where s denotes the sine of the slope of the sewer, and then divest the above expression of its logarithmic form, there follows:

$$d^{10} = 6309574 \frac{A^3}{s};$$

and if the diameter is expressed in feet D, instead of in inches d, we will have $d = 12D$, and

$$D^{10} = 0.0001019.\frac{A^3}{s} = \frac{A^3}{9813.23\, s}.$$

It must be remembered that A here represents essentially a certain volume of water discharged in a certain period of time by the sewer, and that such volume in cubic feet per second is equal to the number of acres in the drainage area when the entire precipitation, at the rate or intensity of one inch per hour, runs off from the surface and reaches

the sewer as fast as it falls; also that if the formula contemplates the discharge of only some fraction of this precipitation, such fraction has presumably been introduced into the constant coefficient. Accordingly, if the intensity y of the rainfall is to be introduced into the formula, the factor A should be replaced therein by the product Ay, which represents the rainfall upon the area in cubic feet per second, thus giving:

$$D^{10} = 0.0001019 \frac{A^3 y^3}{s}.$$

But from the fundamental formula for the flow of water in circular conduits running full, we have the velocity in feet per second: $v = 100 \sqrt{\frac{Ds}{4}} = 50 \sqrt{Ds}$, and the discharge in cubic feet per second: $Q = \frac{\pi D^2 v}{4}$, whence $Q = 39.27 \sqrt{D^5 s}$, and $D^5 = \left(\frac{Q}{39.27}\right)^2 \frac{1}{s}$, or $D^{10} = \left(\frac{Q}{39.27}\right)^4 \cdot \frac{1}{s^2}$. The foregoing two values of D^{10} must, however, be equal to each other, whence

$$\left(\frac{Q}{39.27}\right)^4 \cdot \frac{1}{s^2} = 0.0001019 \frac{A^3 y^3}{s}, \text{ and } Q = 3.946\, Ay \sqrt[4]{\frac{s}{Ay}},$$

which is the Hawksley formula transcribed as above.

In his paper on "The greatest discharge of Municipal Sewers" (Grösste Abflussmengen in Städtischen Abzugskanäle," Zurich, 1880) Bürkli-Ziegler gives the following formula, which is based on Hawksley's expression:

$$q = cr \sqrt[4]{\frac{S}{A}},$$

where q = volume of storm water (litres) reaching the sewer per second from each unit of area (hectare) of the surface drained;
c = empirical coefficient varying with the character of the surface;
r = average rainfall in cubic units (litres) per unit of area (hectare) and per second, during the period of heaviest fall;
S = general grade or fall of the area per thousand:
A = magnitude of drainage area in units of area (hectares).

From the data available, the computed values of c ranged from 0.25 for suburban districts, to 0.60 for thickly populated urban districts, with an average value of $c = 0.50$; and for r it is recommended to take values ranging from 125 to 200 litres per hectare per second. Since 1 litre per hectare per second is equivalent to 0.0143 cubic feet per acre per second, it will be seen that said values correspond to 1.79

and 2.86 cubic feet per acre per second, or to rain intensities of from 1.79 to 2.86 inches per hour. If we take the volumes q and r in cubic feet per acre per second, the area A in acres, and introduce the sine of the general surface slope s in place of the grade per thousand S, we will have: $S = 1000\,s$, and

$$q = cr\sqrt[4]{\frac{s}{A}},$$

where the value of c will range from 1.76 to 4.22, with an average value of 3.52; and if we further substitute the total discharge Q for the discharge per acre q, and replace r by its equivalent intensity of rainfall y in inches per hour, there follows: $Q = Aq$, and

$$Q = cAy\sqrt[4]{\frac{s}{A}},$$

as given above in the text.

The formula of Col. J. W. Adams is developed in his book on "Sewers and Drains for Populous Districts," N. Y., 1880, from the aforesaid fundamental expression for the diameter of a circular conduit running full, viz:

$$D^5 = \left(\frac{Q}{39.27}\right)^2 \frac{1}{s} = \frac{Q^2}{1542\,s},$$

by simply changing the exponent of D from 5 to 6, and then substituting $\frac{A}{2}$ for Q, on the assumption that one-half of a precipitation of $y =$ one inch per hour will reach the sewer during this period of time, thus giving:

$$D^6 = \frac{A^2}{6168\,s}, \text{ or } D = \sqrt[6]{\frac{A^2}{6168\,s}}.$$

For any other value of y than $y = 1$, we would have to substitute $\frac{Ay}{2}$ for Q, thus obtaining:

$D = \sqrt[6]{\frac{A^2y^2}{6168\,s}}$. But for the flow in the conduit we also have:

$D = \sqrt[5]{\frac{Q^2}{1542\,s}}$; and as the two values of D must be equal, there follows:

$$\sqrt[6]{\frac{A^2y^2}{6168\,s}} = \sqrt[5]{\frac{Q^2}{1542\,s}}, \text{ whence } Q = 1.035\,Ay\sqrt[12]{\frac{s}{A^2y^2}}$$

as above given.

The formula of R. E. McMath, of St. Louis, Mo., was published in 1887 by its author in Vol. XVI, p. 183, of Transactions of the American Society of Civil Engineers. Its form is the same as above given, except that for s the fall S in feet per thousand was used. It seems to

have been derived from a number of observations of depth of flow in a variety of sewers of known size and grade draining areas of known magnitude A, but apparently without exact knowledge of the maximum intensity of the rainfall y which produced the computed discharge Q, or of the proportion of water reaching the sewers at the period of maximum flow. The said discharges Q were platted on a diagram as ordinates to the corresponding values of the drainage area A as abscissas, whereupon the enveloping curve of the points thus obtained was drawn and its equation sought. This equation appears to have the form of: $Q = b\sqrt[5]{A^4}$, and by introducing the average surface grade in feet per thousand S as one factor of the coefficient b, we may place: $Q = f\sqrt[5]{SA^4}$, where $b = f\sqrt[5]{S}$; further, the factor f can be regarded as the product of the rate of precipitation in cubic feet per acre per second (or the rainfall intensity y in inches per hour) and the proportion e of water flowing off from the surface, so that we may write:

$$Q = ey\sqrt[5]{SA^4} = eAy\sqrt[5]{\frac{S}{A}}.$$

For the city of St. Louis, Mo., Mr. McMath adopts the following values for the aforesaid factors: $e = 0.75$; $y = 2.75$, and $S = 15$. If, however, it is desired to introduce the sine of the slope s into the expression instead of the grade or fall S in feet per thousand, we must substitute for S its value: $S = 1000\,s$, and by placing $c = e\sqrt[5]{1000}$, there follows:

$$Q = cAy\sqrt[5]{\frac{s}{A}}, \text{ as above given.}$$

For $e = 0.75$, the value of c will become 2.986, which is presumably applicable to first class urban districts; but for suburban districts the proportion e of the rainfall which reaches the sewers is manifestly smaller, and may be taken at about $e = 0.31$, thus giving $c = 1.234$.

A number of other formulas might also have been cited, but as they do not appear to have been based upon trustworthy observations of rainfall intensity, no mention of them appeared to be necessary.

The chief difficulty in the use of the foregoing formulas lies in the selection of the proper values of the rainfall intensity y and the percentage of the precipitation which runs off in the sewer during the time that the rain continues at its maximum intensity; and just at this critical point nearly all writers on the subject content themselves with either complete silence or vague generalities. It may be argued that in the above-mentioned formulas the value of y should be regarded as constant, and that the substitution of any other values thereof than the ones given by the authors will give improper results for Q, owing to the

fact that variability is introduced by the high root of $\frac{1}{A}$ as a factor. This proposition is, however, attended with the admission that the intensity of the rainfall varies with the area, which cannot be regarded as scientific; and the fact also remains that, in most of the cases, the rate of rainfall for which the formulas are alleged to be valid is distinctly specified by the authors. If the formulas are so constructed as to admit of no other rates of precipitation than the ones predicated, they cease to be general in character and become merely particular solutions of the general problem.

It may also be of interest to ascertain which one of the various indexes of the radical $\sqrt{\frac{1}{A}}$ in the above formulas is probably the most correct from a theoretical point of view. For this purpose, let us consider the motion of a material point in sliding down an inclined plane or line whose length is l and angle of inclination x. Neglecting frictional resistances, the time t required for such a point to traverse the length l by the action of gravity alone will be: $t=\sqrt{\frac{2l}{g.\sin.x}}$, where g denotes the acceleration of gravity. If we now regard the length l as the path traveled by a particle of water in its passage from the margin of the drainage area through the gutters or smaller sewers to the point of observation in the outlet sewer, then for different values of l on the same slope x, the time t will vary with $\sqrt{l}$. For similar areas A, however, the length l will vary with $\sqrt{A}$; hence the time t will vary with $\sqrt[4]{A}$; and if it be further assumed that this time t is proportional to the retardation of the discharge, or to the ratio of the sewer discharge Q to the precipitation in cubic feet per second R upon the area A, there follows: $\frac{Q}{R}=\frac{m}{t}=\frac{n}{\sqrt[4]{A}}=n\sqrt[4]{\frac{1}{A}}$, where m and n denote empirical coefficients. Theoretically, therefore, the fourth root of the factor $\frac{1}{A}$ is the most proper one to use in formulas of the class above described; and where deviations from this rule have been made, in order to accommodate the value of Q to certain observations or measurements, it is fair to conclude that the formula cannot be of general applicability.

The most valuable literature on the subject of estimating the volume of storm water in sewers is well represented in the following list: Transactions of the American Society of Civil Engineers, vol. 16, p. 179; vol. 20, p. 1; vol. 25, p. 70; vol. 28, p. 2; also "The Drainage and Sewerage of Cities," by Prof. R. Baumeister. It may be remarked that the subject of sewer construction is not here considered, and hence no reference has been made to a large number of other excellent descriptive articles, reports and works.

E. K.

BRIDGE CONSTRUCTION.

George S. Morison.—M. Am. Soc. C. E.

April 14, 1893.

I have been asked to address you on the subject of Bridges, with no limit either as to the subject or the manner in which it should be considered. It has seemed to me best to consider the subject in a general rather than a special way, and to let my remarks be suggestive of the special points which you are taught in the class room, rather than to give you what would simply be additional instruction of a kind which you are constantly receiving.

In the first place, let us consider what a bridge is. We must look at it as engineers. I am an engineer, you will be engineers, and we cannot frame a correct engineer's definition without considering what an engineer is.

The words engine and engineering go together. The business of a constructing engineer is to make engines, but the word engine in its largest sense includes not only the steam engine, to which the name is usually applied, but every kind of tool, that is, every piece of work the object of which is to do some special work after it is made. A monument commemorates an historical event or marks a boundary; its duty is already done, its purpose is of the past. A tool commemorates nothing, its purpose is future results. The business of architects is to build monuments and the various structures which are more or less allied to them. The business of an engineer is to build tools. An engineer's definition of a bridge must class it as a tool.

And first, what is the great difference which should distinguish monuments from tools? what is the main principle which should distinguish the works of architects from the works of engineers? A monument commemorates what is done, and the fundamental idea is indefinite endurance. The thought of indefinite endurance without rest is torture. A monument should always be in repose and this repose should not be actual merely, but nothing about the monument

should suggest strain. The architect's work should have such stability that there is no possible suggestion of anything else.

On the other hand, a tool is made for work. The idea of work belongs to it. It should be adapted to its purpose, but there is no reason why it should not labor to accomplish that purpose. A strictly engineering structure must always be in a state of strain, and there is no reason for concealing this strain. It is the way in which the tool performs its work.

We begin therefore by defining a bridge as a tool. Every tool has a special duty. What is the duty of *this* tool? It is to carry traffic, sharing to this extent the functions of highways and railroads, being one member of this general group. But it differs from the other tools which carry traffic in that it carries that traffic over a space through which something else may pass. A bridge is a tool which carries traffic over unoccupied space. This is the largest and most general definition; it includes every variety of bridge from the smallest culvert to the bridge which spans the largest river.

This definition however is too wide for general use. Bridges of large dimensions are now being built in cities to span valleys, streets and railroad yards, but this is a specific use and such tools are generally classed as viaducts. For present purposes we will limit ourselves and define a bridge as a tool to carry traffic across a water course.

The bridge tool therefore has two duties: first, it is to carry traffic; second, it is to provide for the water course. It must be so constructed as to accomplish both of these purposes with the best possible results. It must carry traffic in the most economical manner consistent with the free passage of the water course below.

As the object of a tool is to accomplish a specific work it is from its very nature a commercial thing. By doing work it earns money. It cannot be regarded as successful unless it earns enough to pay for itself. These earnings may be in the form of a rental or other direct return, or they may be indirect in the facilities given for traffic which gains without paying. In either case the condition is the same. The construction of the tool is only justified by the tool producing results at least equal to its cost. When a monument is built its cost is planted in it never to return. If a bridge cannot return its cost it should not be built.

Furthermore, the values of different tools vary. It is not always the most expensive tool which is worth the most. The tool which has the greatest earning capacity is the most valuable. The tool which, costing the least, earns the most, is the best investment. Two things

must therefore always be remembered: for how little can a bridge be built; how much can it be made to earn.

But it does not follow that the bridge which costs least in the first instance is the one which will give best results. The profits of a tool are not the selling value of its product, but that value after deducting the cost of working, repairing and maintaining the tool. The selling value of the product is always greater when the product is regular, uniform and uninterrupted than when it is subject to variation and irregularities. To produce the greatest profit in carrying traffic the bridge must be so built that it can carry traffic at all times without interruption from flood or disasters, and carry it in such a way that the public who pay for having this traffic carried have confidence that it will be safely done. It must be so built that it will not require too frequent renewals or too large bills for repairs. In brief it must be a structure which commands the confidence of the public who use it and on which the cost of maintenance and repairs added to the interest on the investment is a minimum. When this result is obtained the perfect engineering bridge is constructed.

A bridge therefore necessarily consists of two parts. One carries the traffic, the other provides for the water-way. In masonry bridges the two can hardly be separated but, as if to emphasize the great difference in its two functions, the modern bridge generally consists of a Superstructure and a Substructure which are radically different in all their features.

In bridges of large spans the Superstructure is usually of metal and frequently designed and erected by men who do nothing else. The Substructure is commonly of masonry, built and frequently designed by men who know nothing at all about Superstructure.

The modern bridge therefore is divided into two parts, the Superstructure, which carries the traffic, the Substructure which provides for the water-way. As the Superstructure must be carried by the Substructure, the Substructure must be put in first and should be first considered.

SUBSTRUCTURE.

While the duty of the Substructure is to carry the Superstructure, the feature which specially distinguishes it, is that it must provide for the passage of the water course. Such provision must be perpetual. It must while carrying the Superstructure provide for the passage of that water course at all stages. It must do more; it must not only provide for the passage of the water in seasons of flood as well as in

ordinary seasons, but it must provide for the passage of all classes of traffic which that water course bears. The tool which carries traffic often has to leave room for traffic below.

Substructure is a subject by itself, and we can only now glance at its most general features.

The foundations must be put in so that they will not be endangered by a maximum discharge. The piers are generally subjected to more destructive agencies than any other part of a bridge; air, water and temperature form a dangerous combination. The permanent bridge pier must not only have a safe foundation, but it must be made of a material which will resist the most potent agencies of decay.

There is another feature about bridge piers; they must not only carry the superstructure but they must allow free passage for the water-way. The piers should be of such shape that they will pass the water with the least disturbance; this is often overlooked.

In the simpler form of bridges the superstructure consists of a single span supported by abutments between which the water course flows. If the bed and banks of the stream are of rock or other substantial material which the water does not abrade, the construction of these abutments is a very simple thing. If the banks are soft and friable the foundations for these abutments may require great care.

If the stream to be crossed is one of considerable magnitude the bridge must be divided into several spans and the water-way divided by piers which support these spans. The form of pier which will pass the water with the least disturbance is a pier with pointed ends and with no angles on the sides, of much the same horizontal section as would be chosen for a ship. A pier with two straight parallel sides tangent to curves which meet in points at the ends, fulfills this condition. A form of pier in very common use has two parallel sides and pointed ends, each formed by two planes making a right angle with each other, thus leaving an angle of 135 degrees at each shoulder. While this form of pier should not be condemned, it does not pass the water with the ease of the pier with curved ends, and when the current is strong a disturbance will be found at both the upper and lower shoulders. In my own practice I always prefer to use the former design, and while in some structures I have made the down-stream end of the pier a semicircle instead of two arcs of larger radius, for many years I have pointed the cut-water and made it of two circular arcs. A pier of this kind not only passes the water well, but, as if to confirm its excellence, it is handsome.

Fourteen years ago I had occasion to design a pier for what was then considered a large bridge across one of our western rivers, and with a

somewhat mistaken idea I tried to make an ornamental pier. When the plans were completed I did not like them; one change after another was made, all tending to simplicity; finally the plans were done. From high water down the pier was adapted to pass the water with the least disturbance; it had parallel sides and the ends were formed of two circular arcs meeting. Above high water the ends were made semicircular instead of being pointed. The pier was built throughout with a batter of one in 24. A coping two feet wider than the body of the pier projected far enough to shed water, and the projection was divided between the coping and the course below. Another coping with a less projection surmounted the pointed ends where the shape was changed. It was as simple a pier as could be built and in every way it was fitted to do its duty. I had started wrong; my work had been done backwards; I had started to make a handsome pier, but did not feel that I had succeeded until I had eliminated everything which was specially intended to make it look well. The pier that was exactly what was wanted for the work was the only one that satisfied the demands of beauty. Forty piers of precisely this design, no change having been made except in the varying dimensions required for different structures, are now standing in ten different bridges across three great western rivers. I have no wish ever to change this simple design.

In designing a pier it must be remembered that the portion of the pier below water has more to do with the free passage of the water than that above water. In a deep river the model form of the pier should begin near the bottom of the river and not at low water. Many rivers in flood time carry a great amount of drift. A pier like that which I have described catches but little of this drift. If, however, a rectangular foundation terminates but little below water, that foundation may both disturb the current and catch drift.

Wherever a rock foundation can be reached without great expense, the piers of a permanent bridge should rest directly on the rock. Where no hard material can be found at moderate depth, the character of the stream must determine whether it is expedient to bear the great expense of reaching this hard material or whether it is wiser to adopt a cheaper substitute. For moderate depths the old-fashioned coffer dam, formed either of a timber curb or of a dam with puddled walls, is still the best practice. Where foundations must be put down to great depths and through material especially permeable to water, the plenum pneumatic process is now in general use. Where the character of the stream is such that the bottom, though soft, is not rapidly washed away, there is nothing better than a pile foundation.

The three principal western rivers illustrate these three methods. On the Ohio River a hard, substantial bottom has generally (though not always) been found at depths where it could be reached by open coffer dams, and this was the only practice until a better acquaintance with the pneumatic process showed that it was a more economical method in many places where coffer dams had formerly been used.

The upper Mississippi is a river with a sandy bottom, generally of great depth, but it has a light fall and an easy current. At two places, Rock Island and Keokuk, no sand overlies the bed rock, and the piers of the bridges at these places rest directly on this rock. In all other bridges across the Mississippi River pile foundations have been accepted as a cheap and safe solution of the case.

The Missouri is a totally different stream from either of the other two great tributaries. It comes through the friable hills east of the Rocky Mountains. It is a silt-bearer of the first magnitude. It may be likened to a great contractor; it has been its business to haul the State of Louisana from a borrow pit in Montana. The bottom of the Missouri River consists of the material which it is hauling down. It is a fine sand, which is constantly changing, and foundations cannot be safely maintained unless they are carried below this silt.

In the two bridges first built across the Missouri some of the piers were put on piles. In all the later bridges the foundations have been carried down below the alluvial silt. In all bridges except the two upper ones, Bismarck and Sioux City, the piers rest either on rock or substantial shale. At Bismarck the piers rest on a hard stratified clay, there being no rock in this part of the country; at Sioux City they are sunk into a gravel which underlies both the alluvial deposit of the river and the high bluffs on shore. In every bridge except the first one built, that at Kansas City, foundations have been put in by the pneumatic process, though at the St. Charles Bridge only one foundation was handled in this way.

The bridge at Memphis is the only bridge yet built across the real Mississippi, that is, across the Mississippi below the union of the four great tributaries. Though the Cairo Bridge is across the Ohio and the Memphis Bridge across the Mississippi, and though the Memphis Bridge is 200 miles nearer the Gulf than the Cairo Bridge, the piers of the Cairo Bridge rest entirely in the alluvial deposit of the Mississippi River and the piers of the Memphis Bridge rest on a hard clay below the alluvial deposit. The Memphis piers are all founded on pneumatic caissons, and in this case a new problem was presented. The caissons had to be placed in water from 40 to 50 feet deep in a strong current.

on a sandy bottom which eroded so rapidly that the caisson could not be grounded before the bottom had cut away to a dangerous degree. To meet this difficulty I decided to carpet the bottom of the river so as to form a substantial surface on which the caissons could be landed. This was accomplished by weaving a carpet 240 feet wide and 400 feet long formed of poles and willows, which was woven floating on the surface of the water; it was then loaded with rock till it sunk on the bed of the river. It formed a perfect protection; the caissons were lowered on this secure bottom and after air was put on the working chamber, the carpet or mattress was cut through and the caisson was then sunk in the usual manner through the sand to the hard clay below. One of the caissons, that of Pier III, a caisson 47 feet wide and 92 feet long, and representing a considerable portion of a pier which cost $300,000, would have inevitably been lost except for this expedient.

Superstructure.

It is hardly too much to say that of late years much more attention has been given to bridge superstructure than to the substructure. Bridge superstructure is preëminently a branch of engineering which can be taught in the class-room.

At the present day the superstructures of bridges of magnitude are almost always made of iron or steel. Perhaps it would be more proper to say, are always of iron, since steel is only a third form of iron intermediate between cast and wrought iron; or more properly, iron manufactured in a peculiar way is called steel.

The manufacture of cheap iron is a comparatively new thing. A century ago iron was not available for bridge superstructures. The bridge tool was made entirely of masonry or largely of timber.

The best way to understand modern bridge superstructure is to study its development.

This development has taken place simultaneously in Europe and in America but on very different lines in the two continents. The development of the European bridge was from a masonry structure to a metallic structure. The development of the American bridge was from a wooden structure to a metallic structure. This was the real order of development though there have been in Europe many noted wooden bridges and there are in America old stone structures resembling those of Europe.

In America, for the first time in the history of the world, a people possessed of modern tools have had at their disposal ancient forests. The steam engine and the saw-mill have made cheap timber from the

forests which in other parts of the world were removed and worked up in early times by slow manual labor. The American builder had at his disposal the most convenient building material ever known. In white pine he had a material five times as strong as ordinary stone and weighing only one fifth as much. The same timber is one-twelfth as strong as wrought iron and weighs only one fifteenth as much. For immediate results nothing equal to it has ever been known. It has however three very serious defects. It is very short lived; if exposed to both air and water it may become worthless in less than ten years. It is very combustible. No additions can ever be made to the original stick and if it is to be used in tension there is a great waste in making proper connections. Its price measured by the unit of strain was formerly hardly more than one-twentieth the cost of iron. The cost of frequent renewals of timber was less than the interest on the additional cost of iron. A structure built of timber after charging up the cost of renewals and allowing a liberal premium for insurance against fire, was still able to earn more money than an iron structure could after deducting interest on the additional cost. In other words, the wooden structure was the most profitable tool. Fifty years ago it was good engineering to build wooden superstructures and it would have been bad engineering to build iron superstructures. Wooden superstructures were universally built. In many parts of the country bridges were built almost entirely of wood; the piers which carried the wooden superstructures were timber cribs filled with rubble stone. In fact, wood was wisely used in ways which at the present day would seem absurd.

Iron bridge building in America really began about 40 years ago, though it amounted to little till after the war. It started on two independent lines, one in the North and one in the South. The real pioneer of iron bridge building in the North was Mr. Squire Whipple. Mr. Albert Fink, then a young engineer on the Baltimore & Ohio R. R., really started it in the South. Both men used skeleton structures and calculated strain sheets much as is done now.

The earlier iron bridges built in the North were really adaptations of the old wooden bridges made in iron. Cast iron columns were used for compression members; wrought iron bars for tension members. The top chord was always to be in compression. The bottom chord was always to be in tension.

The Southern development was of a different kind. It began by substituting iron for wood in some of the tension members, but instead of following the lines of the older bridges and using iron only for the web members it followed the lines of the trussed girder and built up

long spans of combinations of trussed girders. By placing two trussed girders end to end, putting a post under the point of contact and running rods from the foot of this post to the outer ends of the beams the length was doubled and the unsupported length of beam remained constant. This system of construction could be extended indefinitely. In the earlier bridges of this class the compression members were always of wood. In the older all iron Fink bridges cast iron was substituted for timber the tension members remaining as before.

Another adaptation of the trussed girder made the two arms unequal and by combining the beams of a number of trussed girders with unequally spaced posts, formed the Bollman bridge. Twenty-five years ago this was a favorite form of iron bridge. Within twenty years it was universally used on the Baltimore & Ohio R. R.

After the war, iron superstructures began to increase and the Southern and Northern developments met. In both North and South cast iron was generally used for compression members and wrought iron for tension members. The tension connections were generally made with pins but sometimes with screws. The compression connections were square butt bearings.

There was however one exception to this general rule. On the line of the New York Central R. R. riveted lattice bridges, perhaps a higher development of the Towne lattice, but more likely copied from European structures, were used. The first considerable number of field riveted structures were built upon this railroad.

The next step was the substitution of wrought iron for cast iron in compression members. It came first in the long web members and followed in the shorter members of the chords, but cast iron details continued to be used at the joints.

It was at this time considered important to avoid anything like indirect strains. A bridge should be so designed that the strain could be transmitted to each member directly on the line of the axis of that member. About 25 years ago Mr. S. S. Post, a very careful and accomplished engineer, designed and patented the Post truss. It was based on theoretical considerations. It is generally known through the peculiar arrangement of the web but Mr. Post laid special weight on the details. All connections both at top and bottom were made on pins, the only exception being that the several successive panel lengths of top chord butted against each other. The floor beams were hung from the panel points by double loop hangers passing around the chord pins and around pins in the floor beams at right angles to the chord pins. It was admirably designed to relieve the structure of all strains

tending to deform a member, but the stiffness of connections was sacrificed and it would now be considered a very loose structure.

Another feature that characterized the designs of that time was that they were erected without field riveting. The pieces as they came from the shop had simply to be fitted together and screwed up and the work was done.

With the substitution of wrought iron for cast iron in compression members, it becomes evident that cast iron was out of place even in the details of the connections and that the bugbear of field riveting was imaginary rather than real. The cast iron joint box first disappeared in the bottom chord, then in the top chord and finally from the top and bottom of the end posts.

When the fear of field riveting was removed the advantages of riveted connections for floor systems soon appeared. Until about 1873 wooden stringers were generally used on iron bridges. With riveted connections iron stringers came into use and panel lengths increased.

About 15 years ago what is now the general American practice was practically established. Cast iron had disappeared from all truss members. Riveted connections in top chords and floor systems were generally preferred. The Pratt truss or the iron bridge with vertical posts was commonly used and the importance of stiff connections and rigidity as compared with theoretical lines of strain and flexibility was recognized.

Since that time the changes have been rather in the direction of improvement of details, in increased loads provided for, in better material and workmanship and in reduction of strains, than in radical changes of structure. The bridge of to-day is simply the development of the bridge of 1878. The approved practice of 1893 makes bridge superstructures entirely of wrought iron or steel and they are generally rigid structures with little vibration either in the whole structure or the details. The noisy rattle which was very common 20 years ago is seldom heard now.

For short spans plate girders are generally used and the length of this class of structure which was formerly limited to about 40 feet has gradually increased to 100 feet, and will probably go higher as soon as the rolling mills can furnish web plates for longer structures. A few longer spans have already been built and shipped in single lengths from the shops.

In many places skeleton riveted structures are preferred for bridges of from 50 to 150 feet spans. They are commonly called lattice bridges but the name is not strictly correct.

For long spans, pin connected trusses, very different in all other details from the pin connected bridges of 20 years ago, are the general American practice. It is to the principles which should govern the designing of these bridges, that I would next call your attention.

In the first place it must be remembered that while the superstructure rests on the substructure they are very different forms of construction. The substructure is supposed to be perfectly fixed and practically unaffected by either temperature or strain and while this is not strictly true it is practically so.

On the other hand, the superstructure is made of a material which is constantly changing its dimensions. Every change of temperature changes lengths. Every increase of strain does this also.

The superstructure and substructure do not work together. Each must be made as nearly as possible independent of the other.

To secure the maximum of stability with a minimum disturbance the superstructure should be made as complete as possible in itself. It should not depend on the masonry substructure to hold itself together.

In a deck bridge it is possible to put in transverse bracing to the full depth of the trusses. In a through bridge this cannot be done, but nearly the same results can be accomplished by making a deep rigid connection between the floor system and the posts of the web and by strong stiff cross frames occupying all available depth below the top chord. In other words the posts of the two opposite trusses should be so combined that the two together shall form as nearly as possible a single member.

This provision should not be confined to the intermediate posts of the truss. It is specially important that the same arrangements should be made at the ends. Where inclined posts are used this is somewhat difficult but it can be done and the best possible result is obtained by using a rigid portal overhead and by making a rigid connection between the end floor beam and the posts. The old practice of omitting the end floor beam and letting the end stringers rest on the masonry is a very vicious one.

Another thing to be remembered is that the lateral system must have something to pull against. In light bridges it is very easy by over-straining the laterals to produce compression in the bottom chord. In all bridges the end panels of the bottom chord and in light bridges the entire bottom chord should be made stiff throughout. This also forms a great protection if, in case of derailment, the end post is struck.

The independence of the superstructure also requires that it should rest on the masonry in such a way that changes of dimension can be

taken up by free movement and that there will be no undue strains. This matter has not yet received the attention it deserves. While it is almost the universal practice to put one end of an iron bridge on roller bearings, it is usually done in so careless a way as to be virtually useless.

Formerly the end posts of bridges were finished with full bearings which rested at one end directly on the masonry and at the other on a nest of expansion rollers. No provision except accuracy of workmanship, and this often inaccurate, was made to secure an equal distribution of weight. This practice, however, has long passed by and few truss bridges are now built without the use of bolsters in which the spans can rock on pin bearings. This overcomes one of the difficulties but not all. It is a great improvement on the older practice. Expansion rollers, however, are generally so small that friction is unnecessarily great and they are often so placed that they soon become filled with dirt, while the repugnance to the use of cast iron has led many designers to avoid it in a place where it is the most suitable possible material. Many bridges are now built in which the weight is transferred to the masonry through a thin plate of wrought iron or steel with guide flanges at the sides made of angle iron between which are a nest of rollers, which are completely shut in by the guides on the bearing surfaces above and below. The thin wall plates are easily deformed unless set with unusual care and the dirt which gets in between the rollers very soon prevents all movement. The old fashioned sliding plates would really be better than rollers of this kind.

The best practice is to place a heavy wall plate casting on the masonry below the superstructure. This distributes the weight over the pier and keeps the wrought iron members above the masonry where they can easily be protected from rust. It is often forgotten that cast iron is practically an indestructible material while wrought iron and steel are easily destroyed by rust.

Twelve years ago I adopted as a bearing plate for roller bearings a composite plate made of a heavy iron plate at the bottom to which were riveted a series of steel rails, the top surfaces being planed smooth after the whole was riveted up. This forms a stiff bearing plate which is not easily deformed under the weight on the rollers. The dust which collects around the rollers naturally drops into the spaces between the rails, and the rollers can be easily kept clean by working a brush with a long handle from the ends. This plate is made of material which can always be had. I have used it invariably for 12 years.

The general practice in this country is to use small rollers. In Europe,

rollers of larger diameter are preferred and as a sufficient motion is obtained without using the whole circumference of the roller segmental rollers have come into use, that is, the unused portions of the rollers are cut off and the rollers put closer together.

In the Memphis Bridge I found it necessary to use an expansion bearing to roll under a weight of 2,000 tons, probably the largest weight that was ever provided for in this way. I adopted segmental rollers and this bearing rolls on thirty rollers arranged fifteen on each side, the rollers being 15 inches in diameter and spaced eight inches between centers. I used rollers of precisely the same dimensions, but only five in a bearing under the 250 ft. double track spans of the Burlington Bridge which I was building at the same time.

The use of these segmental rollers led to designing an expansion bearing which could be adapted to all bridges, and I now use this special bearing everywhere. It consists, first, of the cast iron wall plate on which is the usual rail plate; on this are the rollers, which are made twelve inches in diameter and placed six inches between centers. On the rollers rests a cast steel bearing plate; the improvements in steel manufacture have placed at our disposal what is virtually a new material. The weight of the truss above is transferred to this bearing plate by a rocker plate; this rocker plate is a steel forging with cylindrical surfaces above and below, the two at right angles to each other. On the upper surface rests the top plate which carries the truss. The rocker plate takes the place of the pin in the bolster but has the advantage of providing for deflection or irregularity of bearings in both directions instead of in one. The combination provides: first, the stiff support of the rail plate; second, a joint which will work perfectly even if set on a surface which is badly out of level.

There are two features of the present practice of bridge designing on which more weight is laid than seems to me consistent with the best practice; in fact the practice of to-day in these respects seem to me to bear somewhat the same relation to the best practice of the future, that the free moving joints and avoidance of stiff connections of twenty years ago did to the best practice of to-day; theoretically they are right, practically they are not important.

The first of these is the use of a single web system instead of a double web system; there is no question that in a bridge with a single web system, the danger of the actual strains differing from the calculated strains is reduced to a minimum, and this is the reason why it is now preferred; it is the best practice for structures of moderate dimensions; on the other hand, in large structures, the single system compels the use of members of such sizes that the connection details are often clumsy

and unsatisfactory; while the opportunity of using members of one web system to stiffen the members of the other is lost; the slight possible irregularities of strain eliminate another source from which greater irregularities are liable to arise.

Another feature of the present practice is the use of curved or broken chords. This has been adopted for reasons of economy and it undoubtedly saves material. With this form of design the single system becomes practically necessary or the web strains may become indeterminate at the points where the lines of the chord change. The reduction of weight is obtained by throwing into the chords a portion of the shearing strain which is ordinarily carried by the web, the chords become of nearly the same uniform size throughout and consequently there is less waste of material in them. The web is very much lightened and the reversal of web strains from a moving load, instead of being confined to a few panels in the center, may be extended through nearly the whole span; this necessarily leads to greater distortion under the passage of a load than with straight chords, as every member is liable to be strained nearly to its maximum by the passage of a single load. Furthermore, it becomes necessary to use adjustable counters throughout as, if the counter strains are resisted by stiffening the tension members, the economy is lost.

The cantilever system of construction has been rather a favorite thing of late years. Many people believe it to be something new, but it is one of the oldest forms of bridge construction.

The cantilever has one great advantage over the ordinary truss bridge. It can be erected without falsework, this being an advantage which it shares with the suspension bridge. Cantilever construction has also the advantage over the suspension bridge, in that it is a rigid structure, this being an advantage which it shares with the beam truss.

Except in those cases where falsework cannot well be used the centilever is not as good as the ordinary truss. The rigidity of all metallic structures is an elastic rigidity. Though a rigid structure its deflections are much greater than those of a beam truss of equal span. Though the weight of a span of cantilever construction may be lighter than that of a beam truss of equal length the additional material required in anchorages and members beyond the limits of the span makes up for any such saving in weight. A cantilever structure is necessarily more complicated than a beam truss and involves features which may cause lost motion and vibrations. Cantilever construction has one advantage; it is possible to transfer the weight of the superstructure exactly to the center of the pier instead of placing it near the edge of the pier as must be done with separate spans.

By using cantilever construction the 450 feet suspended span which forms the central portion of the channel span of the Memphis Bridge, was raised during the high water season of the winter of 1891 to 1892 without falsework. It could have been raised on falsework but it could not have been raised until the fall of 1892 and this delay would practically have made nearly a year's difference in the earning capacity of the bridge. In other words the tool would not have been ready to do its work at the time it was needed.

I have endeavored to touch upon a few of the principal features of bridges and to make my words suggestive rather than descriptive. If any of you desire to build monumental works, do not be engineers; become architects and try to infuse into the architect's profession that knowledge of construction which it lacks. It however you prefer to build the useful appliance which we call tools, remember that it is not merely in the conception of designs or in the general features that the skill of the engineer must be shown; and remember that the whole is never stronger than its weakest part and that weakness is more often found in the details than anywhere else. Never pass anything over because it seems too small but give to each detail the same careful attention that you would give to the largest features.

Remember also that the tools which you build will generally superceed those which some one has built before and are liable to be superceeded by those which a future generation will build; but do not on this account neglect your work or endeavor to make simply the least costly substitute. The users of small tools learn quickly that the cheapest tool is seldom a good investment. The users of large tools are gradually learning the same thing and the constant demand is for better engineering structures.

Above all things make your structures simple. Skill is required to work out complicated details, but it is a much higher skill which makes complication unnecessary. The best teacher I ever saw used to say that if he had but three minutes to solve a problem he would spend two minutes in finding the shortest way to do it. It is said that of the many bridges which cross the Manchester ship caual, all built at once, there are no two alike. This is not the way the best engineer should make his tools.

If you have chosen the engineering profession simply to make money you had better leave it before you begin. The conscientious engineer must take an interest in his work because he loves it. His first idea must be to do good, thorough, first-class work. The pecuniary compensation must be considered simply in the light of compensation and not as the main object of his work.

THE BOSTON WATER WORKS.

LECTURE BY DESMOND FITZGERALD, M. AM. SOC. C. E.

FEBRUARY 17, 1898.

Mr. President and Gentlemen:

It is always a pleasure to meet so many men who are on the threshold of a professional career. This is to be purely an informal talk, so that it will be no interruption for any of you to ask questions.

I came here to-day with some ideas of water supply in my head, but have been so much interested in what has been shown me here at Cornell that if you find me wandering from my topic and talking about the superb views from your wonderful campus and the fine buildings and apparatus on which my mind dwells, you will not blame me for any lack of coherence in my remarks.

We are to look for a few minutes at some views of the Boston Water Works, and I will try to tell you what little I know about them.

It was only a comparatively short time ago that Boston was supplied by one aqueduct 6 ft. 4. in. high by 5 ft. wide, and which could only carry about 20,000,000 gallons of water per day to the city. Now, it requires about 40,000,000 gallons daily to supply the inhabitants. This increase in the rate of consumption is one of the alarming features which as engineers we have to face, and I have no doubt there are some now before me whose energies will be fully taxed in grappling with this knotty problem.

The aqueduct of which I have spoken was built in 1848, and leads from Lake Cochituate, about 18 miles from Boston, to the distributing reservoir in Brookline. When these Cochituate works were constructed the citizens, I think, had the idea that the city would never be large enough to tax them to their full capacity.

So difficult is it to forecast the future. Before ten years had passed it was necessary for the Water Board to issue a notice asking the people not to waste so much water.

One of the most valuable lessons that can be impressed upon the young engineer is that he must plan his works so that they will still form part of some comprehensive scheme even after they are outgrown.

In 1872 the Sudbury river works were begun. This outline plan of the general system of water supply will give you a little idea of Boston and its suburbs with the water sheds we are to discuss.

There are 75 sq. miles in the Sudbury river water shed and 18 sq. miles in the Cochituate, so that you see it takes about 93 sq. miles of territory, to supply the water we need. As we collect in a dry year about 50,000,000 gallons a day from this extent of water shed, in round numbers this requires two square miles to each million of gallons. A long series of measurements has taught us that in New England about one half of the rain that falls upon the ground is evaporated, and this gives us one million gallons a day as the average run-off of one square mile of surface. It takes, of course, an enormous storage to equalize the flow even up to three quarters of the yield of a stream. If any of you care to go into the details of this complex subject, and none will better reward your study, you will find abundant data in the publications of the American Society of Civil Engineers.

This is a view of the streams and towns upon the water sheds. One of the important problems in connection with any extensive water supply system is to prevent the pollution of the water sources. These towns, which you see are mostly situated upon the boundaries of the divide, are sewered upon systems which divert the sewage outside of the water-shed, into gravel or sand beds where it is filtered, and the effluent allowed to pass into streams not used for water supply purposes.

This next plan shows the situation of the storage basins upon the Sudbury river area. We are now building the fifth. The last one, No. 4, which you see at this point, was built by Mr. Fteley, and contains 1,400,000,000 gallons. It covers 162 acres and the water is about 50 feet deep at the dam.

Dam 1, the lowest on the river, is shown you in this picture with a freshet flow passing over the crest. I am sure, after the excellent instruction which you have received, that it will not be necessary for me to call your attention to two important points in connection with dam construction: the necessity for very strongly protected aprons and ample waste way capacity. All of our dams on the Boston Water Works are built to pass safely six inches of rainfall collected in 24 hours. This is Dam No. 2, and the wrought iron bridge upon the crest allows stop planks or flush boards to be placed between the frames. Here is Dam 3. From these two last dams there are 48-inch pipes passing under the bottom of Basin 1 and connecting with the gate house and aqueduct leading from that point to Farm Pond and the city.

Where you have a string of reservoirs, one after the other, this arrange-

ment enables you to draw water from any one of them independently of the others. As troubles are likely to come to any body of water which affect the quality, this enables you to supply only the water which happens to be in good condition, and this brings me to the point of enforcing the necessity of having a great surplus of stored water beyond the actual requirements of a city.

Here is a section of Dam No. 6, now building. In the centre of the embankment is a core wall of concrete 8 ft. thick founded on the bed rock, which in one spot is forty feet beneath the surface of the valley. The up stream portion of this wall is nicely plastered with Portland cement, and upon this plastering we principally depend for the tightness of the dam. For twenty or thirty feet up stream from this core wall the material deposited in the embankment is of a clayey nature; the rest of the dam is of gravel. It is almost needless to say that all the surface soil is carefully removed from the site of the dam before beginning the work of construction. The material is brought in cars into the dam, dumped, spread in layers of four inches, dampened and rolled thoroughly to form a compact bank. I believe that with proper care it is possible to build an earthern embankment 100 ft. high which will not settle over an inch. It takes three or four seasons to finish such a dam as the one you see before you, and the cost is nearly half a million dollars.

The overflow, or waste way, is built around one end of the dam and in the undisturbed soil forming the sides of the valley. These waste ways must be built in a very substantial manner to carry heavy freshets down into the valley safely. The study of freshets and waste ways is an interesting one and you will find a most instructive lesson in the Johnstown disaster so fully investigated and reported upon by a committee of the American Society of Civil Engineers and published in their Transactions.

Before beginning a large dam it is a good idea to put in a small system of water supply for the various purposes required, principally to sprinkle the gravel as it is rolled. A steam pump takes water from the stream above the site of the dam and forces water through a 2½ inch pipe into a tank situated about 150 feet above the valley. Another pipe extends across the dam longitudinally and has frequent connections for hose. This delivery pipe is raised with the dam.

In the first basin built by the city the soil was not removed from the land over-flowed by the water. The result was some very bad water for several years, and I don't know how long it would have continued had the basin not been re-constructed to a certain extent, between the years

1880–84. We learn by failures our most valuable lessons. All of the surface soil is now removed from storage reservoirs lately built or now building by Boston. In this practice I think the city is well ahead of all others. Here is some of this work in progress. You can readily trace the future high-water line.

We have had much experience in this kind of work on the Boston Water Works, and I will endeavor to give you in a few words the methods pursued. The loam is excavated from the portions of the reservoir where the water is more than 8 feet deep, and this loam is deposited in the shallow portions to a height about two feet above high water. This diminishes very much the area to be stripped. In front of these artificial embankments clean gravel is placed forming a facing from two to three feet in thickness. The slopes are generally three to one. Two birds are thus killed with one stone, for the shallow places are cured and the bottom put in clean condition to receive the water. We have many analyses and other observations which prove to us in a thoroughly scientific way the wisdom of this course. It costs generally about 35 cents per cubic yard for stripping. Two of our basins, holding one and a third billions of gallons each, have cost about $800,000 each when completed. These views that are now passing on the screen give an idea of the details of reservoir construction. Here is the method of mixing and depositing concrete. These are the shanties erected by the Italian contractors. Some of their little villages of mud huts are very picturesque.

It has generally been found on our works that it costs about twice as much to do work by day's labor as by contract.

This is a view of a large swamp upon our water shed which contains about 1,500 acres. These swamps are great nuisances to water supply systems. They are prolific sources of color for the water and they give up large quantities of organic matter. I am now working out a plan for the drainage of this swamp, and it is one of the toughest problems I ever tackled.

These are views of the Sudbury river aqueduct, which is 9 ft. wide by 7 ft. 8 in. high and has a capacity of about a hundred million gallons daily. It is 16 miles long. The descent is a foot to the mile. This structure was excellently built under the charge of Joseph P. Davis, when he was city engineer of Boston. In a few places this aqueduct was built upon embankments 50 feet in height and without settlement. I gave you the dimensions of the Cochituate aqueduct, which is shaped like an egg with the large end down. It was built simply of two courses of bricks without any other support, and is now badly distorted in places,

and cracked throughout its whole length. It has been more or less repaired, however, and is now doing good service; but I warn you never to build a conduit for the conveyance of drinking water which is liable to crack where it passes below the line of the water tables in the soil through which it passes, for in this case it acts as a drain for the country and the water it receives in this way by accessions may not always be of desirable quality. The cost of the Cochituate aqueduct, 15 miles long, was a trifle over one million dollars, and the cost of the Sudbury river aqueduct was $3,082,700.

Our distribution system, comprising about 500 miles of pipes in the streets of Boston, has cost about eight and one half millions of dollars.

The additional supply works, embracing the Sudbury system, cost about $6,000,000, and altogether some $21,000,000 are invested in the whole system. The income from this investment amounts to $1,400,000 yearly, which is sufficient to render the water works self-sustaining.

It is, however, very difficult to keep up with the growing demands of a large city. It is something like the growth of a snow ball as it is turned over and over. The only way is never to fall behind if possible, for then the task of catching up seems almost a hopeless one. As the towns surrounding a large city often find it difficult to obtain a good water supply, it seems better to include them in a metropolitan area for certain public purposes, and steps are now being directed towards this desirable end in Boston.

Before I leave a description of the Sudbury aqueduct I wish to say that the original estimate on the Sudbury works of additional supply, as they were called, was $5,000,000, and that they were finally completed for a little less than this sum, though works have since been added, carrying the cost to $6,000,000. Many of the newspapers of the day declared that the works would probably cost $10,000,000 or even $15,000,000 before they were completed; but since this time the citizens of Boston have had more respect for the estimates of her engineers. I cannot impress upon you too strongly the importance of making your estimates sufficient for whatever you may undertake to build. Has not the engineering profession suffered in the past from a lack of courage in this direction? It will depend largely upon you to uphold the dignity and standing of the engineer, and it will be well if you learn early to raise the tone of the profession to as high a plane as possible.

These are views of the aqueduct bridges with sections. You will notice that spaces have been left under the water channel so that the whole length of the masonry can be examined. This bridge at Newton Upper Falls is composed of several stone arches. The longest one has

a span over the Charles river of 129 feet. It is a fine bridge, and there are seventeen echoes under the arch, whence it has been called "Echo Bridge."

One of the essential points in all water works maintenance is to keep every channel through which the water passes as clean as possible. Twice a year we sweep our aqueducts, after emptying them. We have a machine which passes through one portion of our aqueduct. It is very much like a street cleaning machine. It is propelled by the water itself, and large brushes on the sides and bottom revolve against the brick work. The dirty water is carried ahead of the cleaner and is let out of the waste weirs. As the clean water follows the machine, the gates are closed when the cleaner has passed.

We have now arrived at Chestnut Hill reservoir, the largest of the distributing reservoirs. It is in two parts and holds 700,000,000 gallons. The level is 125 feet above high tide. Around this reservoir is a driveway about three miles in length. The views are very much admired, and some attempt has been made to give the surroundings a park-like effect. The water for the supply of the city has been brought thus far by gravitation. About 9,000,000 gallons are here pumped daily to an adjoining hill for the high service supply of the city. This service is of course distributed by means of a separate system of pipes in the streets.

Here is a view of the pumping station, said to be one of the largest and handsomest in the country. I hope you will be able to see it and judge for yourselves. In a very few months there will be a new twenty-million gallon pumping engine erected on this spot which will be a piece of machinery worth seeing. A duty of 140,000,000 lbs. has been guaranteed. As a full description has already been published in the engineering papers, I will refer you to them for the details.

This is a view of our laboratory at Chestnut Hill. In the last few years we have been carrying on some extensive studies into the condition of our water, and our records and researches are largely made and kept in this building. Every week samples of the water are sent to this laboratory from all the storage basins and other sources of supply and here examined. We take samples from the surface, the mid-depth, and the bottom of the water at each point. The life found in the water is then entered in books specially prepared. The number and kinds of algæ growths, the diatoms, the desmids, the infusoria, crustaceans, etc., are all noted, and when more than the normal amount of any kind is found we can shut off the supply from the particular source until the trouble has passed. Photographs are made of forms too

small to see by the naked eye, and I will now show you a series of these beautiful little creations. The results, too, are plotted in graphical form as you will see by these profiles.

Thanking you, gentlemen, for the interest you have shown in these pictures and your patience with me in describing them, and trusting that some whom I now address will have the good fortune to devote their lives to the branch of the profession which I have feebly attempted to illustrate, for I know of none which brings greater good to our fellow men, I will now bring this rambling talk to a close.

THE CONSTRUCTION OF HIGHWAYS.

By James Owen, M. Am. Soc. C. E.

April 21st, 1893.

I made the suggestion to your Professor a few minutes ago, that instead of delivering what I would call a formal address to you, I would make this a sort of a talk between us, and to that end I would suggest, if you gentlemen are willing, that if there are any points about which you feel doubtful you would ask questions. Much more is learned sometimes by asking questions than by explaining the matter itself.

The first thing I want to impress upon your mind on the road question is that it differs considerably from engineering itself. Road construction is subject to engineering views that may be modified by political views. Almost everybody in authority thinks, in fact knows, that nothing can be taught on the question; and the first thing to impress upon his mind is that he does not know much, and second that *you do.* That is a very important particular to remember. Let your client know that you know your business, then, when that point is gained you can build good roads. There are no *business* requirements in road construction. It is purely a question of outlay of money; no gain is intended, and to get the best returns from that outlay, both in present construction and future returns, is the problem.

In dealing with the road question, the first thing that arises is the location. I have found, and you will also probably find, that very little of your practice will be on the question of location. You will not build many new roads, but will improve the old ones. If you have occasion to work in that line you will find the legal problems will interfere with the engineering problems. You will find in road location there are other questions to be decided beside engineering problems, and you will find harder work than you think. There is one question, however, that comes up, and that is the re-location of old roads. In mountainous countries the roads are too steep. Out west the roads are rather long, and there the effort now is in shortening distances. We must consider, therefore, these two points: Easement of grade and shortening of the

distance. Locate it with the easiest grade and with shortest route. The balancing of the two should be the ultimatum to be gained.

We have gotten the location, and we have now come to the question of the establishment of grades, and here you will find more trouble. I wish to state here that my idea of the standard grade of a road is one foot in a hundred. A departure from that is to be resented. The maximum limit of a grade for ordinary travel is four feet in a hundred; nothing more should be allowed if avoidable, and nothing over ten per cent. under any circumstances. The least grade that should be permitted is one half of one per cent.; water will flow at that, but less than one half of one per cent. should not be allowed. These are standard rules.

The question of "*cuts*" and "*fills*" in road work is governed by different influences in different roads, and differs also from railroad work. In railroad practice cuts should balance fills. Of course, in allowing these cuts to balance the fills you allow for shrinkage. That is to say, you may have 15% more cuts than you do fills. In road-building the balancing of fills can be used only in rural regions. The farmer does not want too much of the "fills" on his ground. When you arrive at the point of establishing grades on suburban property, you will find that the cut is the ideal of standard construction in the suburban location. This is the height of the suburban development; hardly ever make any fills. I find it is much better for everybody interested. In urban or city property economy dictates the opposite rule. For the best advantage of the lot the street wants about five feet fill. When you dig a cellar you have then enough earth to put the lot on grade with the street. In establishing roads and streets in rural and suburban districts I have eliminated as much as possible the straight lines, and observe the lines of nature; the appearance is far in advance of the absolutely straight line. No such a thing exists in nature as a straight line. The thing that comes nearest to a straight line which I have found is the face of a crystal.

In laying out curved roads with these grades you will find that when you have laid a grade on paper, and when you come to glance over that grade in development, corrections have to be made for appearance sake. Once, in building a road through one of the parks of New York city, the chief engineer came along and asked who laid that grade. Well, I happened to be the unfortunate man. He said, "it is lower on one side than on the other." To save my reputation I took the level and ascertained that the road was exactly the same on both sides. This shows that the level sometimes must be sacrificed for the sake of appear-

ance. Now we have our road located, we have its grade, and we next come to the crowning of the road. I will give you the figures as practiced in our section: Thirty feet, ten inches; thirty-five feet, twelve inches; forty feet, fourteen inches; fifty feet, eighteen inches. *That* crowning has been arrived at by experience, and gives good success. Engineers are apt to give too little crowning. These figures are a fair compromise, and will give good results. Remember this, though, that crowning of a road never remains of the same height at any one place. In the first place, when you build your road, the consolidation is more at the center than at the side. Then the road wears down and your crowning wears out, and you have a continually fluctuating line. Never build a road level in the center; then always give it if possible a drainage. I have created artificial grades for that purpose. If you build a level road travel in the center will form ruts and cause the water to stay there. Then, after you have got the crowning established, comes the construction of facilities for drainage. Now I can state that in roads with grades of four feet in one hundred, ample drainage is secured, and the building of artificial drains is unnecessary. If you have grades flatter than that, places where the water will probably stand, it is best to build drains, longitudinally or horizontally. An old-fashioned drain trench consists of a simple trench in which is laid a two inch tile, well packed with hay and straw; the other drain is made by placing broken stone in the bottom of the trench and brush on the top, then earth. Blind drains are good, but they are apt in the course of twenty-five or thirty years to become choked up. I substitute a form of a drain which has the tile in the bottom, then the stone, covered by brush and earth. When you have a tree near the road its roots will go for the drain, so by doing this I find the roots start for the pipe, and grow in the stone above but do not find the drain. You must understand that you do not want to spend too much money on drainage, and only do it where you cannot help yourself. When you are constructing roads do not imagine you have got to have a system of drainage. Incidentally a question of culverts may enter in, and I will give you a few points on this. My early practice was to lay ordinary tile pipe; I found that if I put them too near the top there was trouble. Now I substitute cast-iron pipe. We buy rejected pipe. After I go over a foot in diameter I generally put in flagging. The thickness for three foot span should be four inches; for four foot span five or six inches. If less than this thickness I have found flag to break in exceptional winters. I have substituted for over four feet a structure made with stone abutments, and put in iron beams and brick arches between the beams and levelled

up with concrete, and on top broken stone. You will find this absolutely permanent.

What you want to do is to make something that will stay.

Now I will pause, and if any gentleman has any question to ask now is the time.

Question. I do not quite understand the construction of those arches.

Answer. Up to eighteen feet rolled iron beams go across, and in the opening between are the arches. Put nine feet apart. Fill up with concrete. After twenty feet the rolled iron beams are too light, and I generally then put in something heavier. Now they make thirty-inch beams.

Q. In country roads, where these arches are too expensive, what would you use?

Ans. Use dry soil, stones or timber. I find that in the country they are as anxious to have the best as in towns.

Q. Are there any tie-rods used?

Ans. Yes.

Q. Do two-inch drains ever choke up?

Ans. No. That last drain with the tile pipe and stone above I have success with.

Q. Are these pipes in the center or side?

Ans. In the side.

Now we come to the road question itself. We have got our grade, we have got our line, we have got our trench and ideas of form. Now for material. Generally speaking, road material may be classed under the following: sand, clay, drift, black stone, turf, shale, gravel and field stone. From that list we can mark the first four as things not to be desired. Mark them off the list and say we do not want them.—I have known shale roads built that were very successful however. The next natural product used is gravel, and you can say this of gravel, that the more clay there is in it and the quicker it makes a road the sooner it wears out; the more siliceous material and the larger the size, the longer it takes to make a road and the better it wears. You can roll it and it takes two or three months to make, but it will be a good road ten months in the year. If you want twelve feet of gravel put it on about eight feet and it will spread to twelve. If you put it over the whole it will spread so you will not get what you want. If you have gravel with large stones and small stones, have the large stones as much as possible at the bottom.

After these general remarks we come to the question of artificial roads. At this point I propose to eliminate the question of city streets.

One thing is necessary in this work, and this one thing is money. The next thing is, ways to get it.

With regard to country roads and suburban roads and side streets in large cities and main streets in small cities there is no question but what satisfactory results can be obtained by the macadam pavement road. I am not a "thin road" man and when I did make one I made a dismal failure and do not want to try another.

Draw your specifications for your road and build it strictly in accordance with those specifications. That is the engineering view of road construction, and you will have that question always before you. The people who are raising the money want it to go as far as it will stretch.

This gives you a rough idea. First bring the road bed to the same form as the finished surface; grade to desired shape. The next question: some say, roll the bed when you have got it to that form. I say, do as you please, and I find that we can get along without it. Now lay your foundation. Do not use any stones for this which are affected by frost nor any stone that is round.

Size and depth of foundation. I use for an 8′ road 5″; for a 10′ road, 6″; for a 12′ road, 8″. The theoretical way of laying these stones for the foundation would be to have the large flat part of them at the bottom so that the broken stone used for filling shall wedge the foundation tight and keep them from working to the surface, which would naturally be the same with *round* stones. The whole thing is now hammered until it is uniform in surface, and then finished. Wedging is the secret. If you only get the foundation wedged it is immaterial how it is done.

After this put in packing. Some engineers insist that no packing should be used, but good results with much less money can be obtained with packing. Ordinary loam, not the top-soil, is used in packing. Clay can be used but sparingly; sand and gravel can be used dependent on the weather and time of year. You can use more packing in the spring with a probability of dry weather than in the fall with the probability of wet weather. Do not put on too much, but sufficient to see the tops of the stones protruding through the soil. Never have your packing dumped on the road but always spread it with a shovel because the result will be that where it is dumped you will find that there will be more in one place than in another. After the packing is on then comes the roller. Up to 1878 I put a clause in my specifications to use a steam roller, but now we use a horse roller. When you realize that highway bridges built ordinarily for one hundred pounds per square foot you will see that three hundred pounds to the sq. ft., such as the

steam road roller gives us is loading too much, so I abandoned the steam roller and adopted the horse roller. The solid cylinder of iron is another good point of this roller. I use this kind, and if I want to get an extra heavy one I fill it with water. Now for the broken stone. The material we use in New Jersey is one, and one only, and it is the best that can be found. Trap rock. It exists in abundance in all Northern New Jersey. It is cheaper to bring trap rock one hundred miles than to use granite in site, and also to haul it two hundred miles than to use limestone in site. This comparison is based on the assumption that the railroad delivers the material directly to the place where it is used. The main expense of transportation is not caused by the coming of a long distance by railroad but in being transported by teams from the railroad to the place where it is used. Now we will assume that we have trap rock. What size? Practice says that the size best adapted is the one and one-half inch. Get it when you can. I had trouble at first in getting it because the crackers are not constructed to produce that size, so I have eliminated entirely the one and one-half inch in my twelve inch specifications.

I will state here for your information that the price of stone has been reduced in the last few years so as to create a new era in road building. The same road can be built for one half the price now that it used to cost.

Remember you want to allow two and three quarter inches, of one and one-half inch stone, to produce two inches. We have our foundation and packing, and packing rolled. Then I work my road in this way. On top spread three and three-fourth inches of broken stone. Roll this broken stone. I do not keep the roller idle at this point but send it on on after it has consolidated the broken stone.

Now the next stretch of the road which has already received the packing and from this on to the next which has received the layer and thus pack and consolidate the different layers of the work as they are laid down while the roller is kept busy all the time. At the same time if possible I let the travel come on at once, it packs the bed as much as anything. Let the travel come on, and all the time the stone is being more consolidated. Sometimes after two days a wagon can drive over the surface without perceptibly marking it by the wheels. After I get the stone consolidated I put on my stone dust. If the road bed has not been thoroughly consolidated the stone dust disappears. That is one grand gain of the packing, for if it is not used the road works upside down. A road can be built cheaper with packing and a horse roller than by not packing and using a steam roller.

Q. How do you keep it from spreading on the outside?

Ans. I put the road in a cut if I can. Do not put the pavement, on the new road, on the surface. Dig out of the ground to the depth of your pavement.

Q. What is the use of stone dust?

Ans. It is a good deal like the blacking on your shoes. Very nice but you can do without it. I am seriously advocating making roads without stone dust at all.

Q. How long will it last?

Ans. Varies with travel. I saw one road that was built nineteen years ago that was very good.

Q. What is the composition of this trap rock?

Ans. It is an igneous rock and composed of different materials such as feldspar, etc., but no mica.

Q. What are the relative merits of the surface curved throughout, and composed of two planes.

Ans. You do not have advantage of appearance to begin with. I do not like the appearance. There is no strong technical reason that I can give.

Now there is one other point I want to allude to here. It is a vital point. That is repairs. Under this description I have given, you have got your road, and if you have seen that the work has been done right you probably have a good road. You must not think that the road will not require repairs although the public think so.

The demand is unavoidable but the difficulty is to appreciate the necessity to raise money for the repairs. As to the repairs themselves I differ very much from the maxim of the old road builders. They said you should have the stone and men on hand to repair any place as it needs it. Theoretically I say the road should be uniform and should wear uniformly. Consequently it should be your ideal never to require patching. My practice is to take a section of road, my broken stone, the packing stone and dust, and lay on in the same manner as in making it. In the interest of economy I eliminated dust except in places, but found that I could repair roads merely by using broken stone and packing and not using dust. I think it is possible that you can build a road without using dust, and I know you can repair it without using dust. The construction is merely incidental.

I would state this, that in France the old maxim was followed of having stone and man on hand when needed, but I find now in the report on the "Chaussees" that whole sections are now taken for re-

pairs, and this method is virtually the same which I follow by "natural selection" if you choose to call it so.

As I said in my opening thought, road work is purely political work, and engineers will have to be men, good and strong, who can do their work well. Appreciate these things, in your practice, and be loyal to yourself, your associates, and your Alma Mater.

Railroad Accidents and the Means of Preventing Them.

Professor Crandall, in introducing the speaker, said, "The lecture this afternoon will be by Mr. H. G. Prout, Editor of the Railroad Gazette, who will speak on the subject of railroad accidents, to which he has given a good deal of study and investigation, some of the results of which he will give us."

The lecture was illustrated by numerous stereopticon views which cannot be reproduced, and therefore, we have taken the liberty of abridging it very much to adapt it to publication without illustrations. The description of various mechanisms could only be made intelligible by illustrations and therefore most of it is omitted.

Mr. Prout said "When Professor Crandall did me the honor to ask me to speak to you to-day, I naturally cast about for some subject which would be of interest to you. I reflected that my audience would be composed of the future Chief Engineers, Superintendents, General Managers and Presidents of the railroads of the United States, and it struck me that it might be interesting, and perhaps instructive to you, to know something of the causes of railroad accidents, their cost, and how they may be prevented.

"The first great point of interest in railroad accidents is the money cost. It is very difficult to get at statistics of the cost of accidents, but from the accurate returns of two or three roads, I estimate that the total cost of train accidents in one year is about $9,200,000 in the destruction of railroad property alone. Another serious element of cost which cannot be accurately estimated, but which probably amounts to at least $3,000,000 a year in the United States, is compensation for death and injuries to persons. So we have over $12,000,000 a year as the cost of railroad accidents and this does not include the destruction of merchandise, damages at highway crossings, and various other matters. You will see therefore that the money total is enough to make the subject one of great importance.

"The humanitarian side of the question is perhaps even more important. The statistics of 1891 show that in that year 6,334 persons were killed on the railroads of the United States and 29,025 injured; that is

6

about 800 more persons were killed and 3,000 more wounded than were killed and wounded in both armies at the battle of Gettysburg. These are truly appalling figures, and if taken alone would naturally make one hesitate about taking a railroad journey, but of all the persons killed in the United States on all the railroads every year only about 300 are passengers, while the total passenger movement in the year 1891 amounted to over 13,317,000,000 passenger miles. In my own business I travel about 8,000 miles a year, and it is a matter of simple arithmetic to conclude that I must travel at this rate incessantly 555 years or thereabouts, before I am killed in a train accident. You will see therefore that there are some consolations in statistics.

"We divide railroad accidents into train accidents and accidents other than to moving trains. The train accidents include derailments, collisions and a few others, and it is these that we shall consider this afternoon."

Here the lecturer showed a number of slides, exhibiting collisions, showing effects of telescoping and the like, and explained the immediate causes of the various accidents such as failure to read signals properly, failure to transmit or read orders correctly and so forth.

"There are two principal methods of preventing collisions: First, the rules and methods of operating, and second, mechanical safe guards such as brakes and signals. In the United States the power brake has reached its greatest perfection, but much the greater part of the trains in this country are run without other mechanical safe guards. The rules of operation are the chief reliance. Trains are scheduled to run at certain intervals and to meet and pass at certain places. The time tables show the rights and duties of each train as related to other trains, and if all trains were on time and there were no extras it would be a simple matter to prevent collisions; but the conditions are not so simple. For many reasons extra trains must be run, and the emergencies of a great and complicated business constantly throw one regular train or another out of the scheduled time. To meet these emergencies and prevent the hopeless delay that would follow the attempt to move numerous trains by printed time tables, some man must constantly interfere with the prearranged order of things. That man you find on all the divisions of all the railroads of the United States. He is a clear headed, strong nerved man, and he sits at his desk watching scores of trains scattered over hundreds of miles of track and sending out the orders by which they move. If he makes a mistake someone gets killed. But his swift and incessant work must go on. He bears a load of responsibility that would kill you or me in a week, but he thrives on it and you can hire

him for $75 a month. That man is the train despatcher and he has my profound admiration and respect. And just here I wish to say a word that will be worth more to you than all else that I shall say this afternoon. Many of you will be surprised when you get out to do your work in the world, to see what an amount of pluck and sense and judgment and devotion to duty there is among the greasy fellows who would run a mile to dodge an equation of the second degree, but it is these high human qualities after all that compel success, and chief among these is pluck.

"The orders sent out by the train despatcher are repeated back to the operators as a safe guard against errors in transmission. The time table and the train orders issued by the train despatcher are supplemented by the rules which require trainmen to protect their trains by flags and torpedoes in cases of delay. On all roads the trainmen are required to flag their train in all but scheduled stops, and on most roads a flag must be sent back if a train is losing time after its speed is reduced to a certain limit.

"If the rules were always obeyed and if no mistakes were made there would be no need for mechanical safe guards, but accidents from negligence or fault of the employés are among the most common. In fact in all the train accidents of the last five years 55 per cent of the deaths and injuries to persons have resulted from accidents due to negligence in operation." Here the lecturer cited a number of instances of serious accidents caused by different kinds of mistakes and carelessness.

"Among the devices to secure safety the air brake stands first. The modern air brake has reduced the distance in which a stop can be made to about one-fifth of the distance required to stop the same train by hand brakes. In the Burlington brake trials for instance, it was found that a freight train of 50 cars could be stopped by air brakes from a speed of 40 miles an hour in 600 feet. The same train, under the same conditions, could not be stopped by hand brakes in less than 3,000 feet. At 20 miles an hour an air braked train can be stopped in from 75 to 100 feet. Good express train practice could make a stop from 40 miles an hour in about 500 feet, all the conditions being right; but the difference between the quick stop and a slow one is often life and death." Here other accidents were cited showing the very narrow margin of distance which decided whether or not a fatal accident should take place in some recent cases. "You will understand therefore the power of the air brake as a tool and why I have said it ranks first in safety appliances.

"The next great important feature of the air brake is its automatic function; that is, the quality by which the air brake stops a train in

case the train is broken, or the brake pipe parts. It is this feature also that allows the brake to be applied from any part of the train." The lecturer then explained in detail the arrangement of the parts of the air brake apparatus on a train and the action of the triple valve, pointing out how by storing the air pressure under each car in the auxiliary reservoir the time required to apply the brakes was greatly reduced from what had been required in the old straight air brake. For instance, the brakes can be made to begin to apply on the rear of a 50-car train in two seconds. A 50-car train is about one-third of a mile long; that is, the impulse which causes the application of the brakes travels at the rate of about 600 miles an hour.

"One of the most interesting brake problems to-day is that of stopping from high speeds. For a good many years it has been well-known that a given brake power does not produce as much useful effect at high speeds as at low speeds. This is leaving out the question of the greater energy of the faster train, and considering merely the question of the retarding effort delivered on the wheels. With the same power in the brake cylinder the retarding work on the fast wheel will be less than on the slow wheel. This is chiefly because the coefficient of friction is less at high speeds. It follows that if your brake pressures and leverages are adjusted so as to get the maximum effect short of sliding the wheels at 40 miles an hour, when the speed gets down to 30 miles the wheels will slide. Then you will not only ruin your wheels but you will not make so good a stop. The perfect brake then is one in which the shoe pressure will diminish as the speed diminishes during the stop. The importance of a perfect brake is becoming more and more apparent every day as speeds increase. You can stop from 60 miles an hour in a thousand feet if you have brakes on all the wheels, a dry rail and are running on a level. The chances are that it would take you 1,300 or 1,500 feet to make the stop; but supposing you have occasion to make a stop from 90 miles an hour you will find that at the end of 16 seconds your train will have run 1,800 feet, and it will still be running 60 miles an hour. It would hardly be possible to stop from 90 miles an hour in less than 3,000 feet, and the chances are that you would run nearer 4,000. But speeds of 60 miles an hour are made every day, I might almost say every hour in this country, and such speeds are reached on almost all of the great railroads. In fact there are several trains running regularly that are scheduled up to speeds that require them to make more than 60 miles an hour in certain parts of the run; and several records of over 90 miles an hour have been made. It will be obvious to you therefore that the importance of a perfect brake cannot

be over estimated. Various devices have been tried for automatically reducing the pressure on the wheels as the speed falls, but none of these have yet come into practical use. The Westinghouse people have been for some months experimenting, with very encouraging results, on a device for accomplishing this end, and there is, I should say, a fair chance that it will soon be brought into use. What I have said will indicate to you a very interesting and very important direction in which to investigate in case you should have occasion to make a special study of the air brake.

"Perhaps the greatest advance towards safe running that will be made in the next decade, by the railroads of the United States, will be in the protection of trains by signals. For 20 years there has been a slow progress towards uniformity in signaling. In the last 10 years this has been very rapid." Here the lecturer showed slides showing the various standard forms of signals and explaining particularly the semaphore, which is now the most generally accepted form. He then went on to speak of interlocking signals and of block signals.

"Fixed signals have two functions. First, to protect junctions and crossings, which is interlocking; second, to keep trains apart when they are running on the same track, which is block signaling." The steps towards interlocking switches and signals were then briefly explained. "That is, the operation and management of the signals, protecting any given point were concentrated in one place and the levers operating the switches and signals put under the charge of one man. Then mechanical means were devised by which the movements of these levers could be made to depend upon one another in such a way that it would be mechanically impossible to move them except in a certain sequence. For example: supposing you wish to make a route for a train through a junction. The first step is to set the signals against all opposing trains, then to set the switches in the proper position for the track to be used, then to clear the signals permitting the train to move over the route which has been made. The levers by which all of these operations are performed are interlocked in such a way that this sequence of motions must be followed and that no other one is possible." Here slides were shown illustrating various junctions and yards and the interlocking frames containing the levers by which their switches and signals are moved, and an explanation was given of the means by which the levers are interlocked.

"By block signals a fixed interval of space is maintained between two trains running on the same track. This system is practically universal in England. The train order system which is almost universal in the

United States aims to maintain between trains an interval not of space but of time. It is obvious that it is assuming a good deal to suppose that two trains that start 10 minutes apart, are actually 10 minutes apart after the lapse of half an hour, and the time interval is a very dangerous reliance. On the contrary the space interval, if maintained absolutely by block signals, gives perfect security against rear end collisions. The simplest form of block signalling is that by which the road is divided into sections of say two, three or four miles, according to the amount of traffic passing over it, with an operator at the end of each section whose business it is to stop trains or to allow them to proceed by the display of proper signals. These stations are connected by telegraph and the operators communicate with each other by a simple code of bell signals or, in some cases, by the Morse instrument. Thus a train is not allowed to pass one block station until word has been received from the next one in advance that the preceding train has passed that station and, theoretically, an interval of two, three or four miles is is always maintained between two trains moving in the same direction.

But under this system accidents sometimes happen because the first operator allows a train to enter a block section before he has received word from the secord operator that that section is clear. Collisions also happen because the block system is worked "permissively" as it is called. That is, the second train is held at the block station for a certain number of minutes and then allowed to proceed into the section "under control," the engineman being warned to expect to find a train ahead of him before he reaches the next block station. Against the dangers of permissive blocking there is no safeguard; the only remedy is to make your blocks short enough so that you can work them all "absolutely;" that is, never to allow a train to pass into the block section until you know that the preceding train has passed out. But mechanical provision can be made and is made against an operator making the mistake of lowering his signals before he has received word from the signalman in advance that the section is clear. This is best and most efficiently done by what is known as the Sykes system of interlocking two or more block signal stations. Under this system, when the signal at A has been put to danger it cannot be cleared without the permission of B. It is held at danger by an electric lock, but B cannot give this permission until his own signal has been put to danger. The sequence of operation then is this: A train passes A and is protected by the signal put to danger behind it. When the train passes B it is protected by that signal and when this is done B can unlock A's signal and allow that signal to be cleared for another train. Thus there must be the

consent of two men before the signal at A is cleared. It is not impossible for both of these men to make a mistake and for B to put his signal to danger before the train has passed out of the section and to unlock A's signal allowing another train to enter. This is provided against by an electric apparatus operated by the train through a short piece of track circuit at each block station." Plates were exhibited showing the Sykes apparatus and the operation of the mechanism was explained in some detail.

"The great obstacle to the use of the block system is its expense. If the block stations are numerous the wages become a burden. If the block stations are not numerous the movements of track are interfered with. Therefore, it is important to make the blocks as short as they can be without unreasonable expense, and this leads to the automatic block system by which operators are done away with, and there is no expense other than the first cost of putting in the signals and the cost of keeping them in repair. The automatic block signals which are used on ordinary roads running heavy and fast trains, depend for their operation upon electricity. A mechanically worked automatic signal is used to a slight extent upon the Manhattan Elevated Railroad in New York and the Staten Island Rapid Transit, but the applicability of this method of operating automatic block signals is quite limited.

The lecturer then showed lantern slides illustrating the Hall wire circuit system and the Westinghouse electro pneumatic track circuit system and explained the operation of these signals. And he closed by impressing upon the audience the fact that when all is done to secure the best mechanical appliances the final reliance for safety must be upon the discipline of the men, and the railroad officer who makes the greatest success in the operating department and commands the highest salary is one who can best control and manage his subordinates, and no great success can be expected in this department of railroading except by one who has developed the organizing and executive faculty.

NOTE.—A very clear explanation of the air brake in its latest form may be found in the recent editions of the "Catechism of the Locomotive." The Westinghouse Air Brake Company issued a good iustruction book fully illustrated which can probably be obtained for the asking. For an elementary but clear explanation of the principles and practices of interlocking and of block signaling the reader is referred to Barry's "Railway Appliances," published by Longmans, Green & Co., London; the catalogues of the Union Switch & Signal Co., Swissvale, Pa.; the Johnson Railroad Signal Co., Rahway, N. J., and the Hall Signal Co., 50 Broadway, New York, give much information of value. The Hall catalogues are especially full. "American Practice in Block Signaling," published by the *Railroad Gazette* will be found to give excellent descriptions of the most recent block systems, including the Sykes apparatus.

AMERICAN METHODS OF BRIDGE ERECTION.

*A LECTURE DELIVERED BY

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BEFORE THE COLLEGE OF CIVIL ENGINEERING,

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Fellow Students,—You Cornellians who preceded me, and you majority that succeeded me in these dear halls, I greatly prize and appreciate the privilege accorded me of being with you to-day. Among the enviable advantages that have been fostered and multiplied for you by the conspicuous devotion, energy and ability of the Dean and faculty of this college, many new features have been developed in the nearly twenty years that have elapsed since my matriculation.

Among them is that of "Experience Talks" from practicing engineers, some eminent men of whom have recently given you what were substantially epitomes of their own difficulties, failures, successes and practical expedients.

Now these, especially the former, are the most valuable points that one engineer can offer to another, and for the exchange of which chiefly the eminent brotherhood of the American Society of Civil Engineers exists.

You undergraduates have here the most perfect and complete of technical curriculums, munificently equipped for practical field and laboratory work and well designed to lay the broad and deep foundations of mathematical and scientific preparation which is the real essential of "An Engineering Course." Your time here suffices only for a general technical equipment, to learn where to study and how to study, and these more or less informal lectures serve chiefly to foreshadow to you the practical conditions and methods that arise in various constructions,

and to present the horizon where each should select his distinct goal and patiently pursue it with the endless zeal and application of the specialist, for, of all men, the civil engineer should know something about everything and everything about something; this afternoon we will have something about one thing, namely, American Practice in Bridge Erecting. And I especially regret that the illustrations of it are so few of them of work with which the speaker had direct responsibility or personal experience. The general classifications and considerations are of the fundamental and elementary field operations and are in accordance with some years of active work in shops, field and designing room, and the practice of prominent erectors. The illustrations and descriptions of characteristic and notable erections have been prepared from personal notes and sketches of important works in progress constructed during recent years, largely from original drawings of false work, travelers, etc., from special study of erection methods and operations in general, from verbal descriptions and explanations by practical men, from the very few publications of technical societies and from current accounts in various technical periodicals, notably *The Engineering Record,* which alone has made a special department of this great branch of engineering, and in whose service I had collected much of the data.

Bridge erection, in a broad sense, includes the assembling in place, connection and adjustment of almost all framed and trussed structures, chiefly bridges and roofs, either permanent or temporary, primary or auxillary; but in this part of this country, and as the most developed art, it refers chiefly to large structures composed of iron or steel members, with which we may properly deal from the time they leave the manufactory until the final inspection and acceptance by the purchaser's engineer.

The subject has three principal divisions: First, Primary Structures, usually permanent; second, Auxilliary Structures, usually temporary; third, Working Plant.

Bridges may be assumed to include all structures designed to transmit strains of flexure to relatively solid seats, and thus embrace roofs, girders, highway and railroad bridges, viaducts, aqueducts, towers, columns and wind and crane bracings that form most of the first division. Primary structures may be either simple or compound; a simple structure practically consisting of a single piece, as a column or plate girder; simple structures may be either directly placed or temporarily supported.

Compound structures may be very elaborate, like the complicated trusses of a long-span railroad bridge, and are essentially structures formed by assembling several members or parts delivered separately at

the site. Compound structures may be, during erection, naturally self-supporting, artificially self-supporting, or non-self-supporting. Non-self-supporting structures may be erected on the ground or on falsework.

Auxilliary structures are chiefly designed solely for the erection of permanent constructions, of which they may serve the whole or portions; they may be fixed or movable. Fixed structures include trestling, towers, piles, framed trusses and suspended platforms. Movable structures include shear-legs, gin-poles, derricks, rolling towers and platforms and boats.

The present American practice is notably superior to the foreign in the completion of members by power tools in the manufactories, their design with special reference to rapidity and accuracy of field assembling and completion of joints, and for the liberal use of special engines and steam and hydraulic power in the field that was promoted by the magnitude and economy of American work.

In the admirable monograph on American Bridges, presented by Theodore Cooper to the Am. Soc. C. E., the development of long spans and consequently of heavy members and difficult erection problems is traced, and it is shown that, except some moderately long timber spans, no great and heavy trusses existed until recent years, so that their erection is the art of this quarter century and its most able masters are of the present generation, who have created methods and appliances at least as fast as the designing engineers and manufacturers have furnished them with structures of increasing proportions to handle.

The first wooden bridges were doubtless built on continuous timber scaffolds, each moderate sized piece being framed on the spot and readily placed in position by hand tackle, levers and skids; and as the light highway iron work of twenty or thirty years ago was introduced, old methods were modified to suit. When railroad bridges became important, the erection of almost every structure of magnitude was a problem requiring special solution, and new methods and tools have been constantly devised, modified and perfected until the mechanical and constructive skill, ability and facilities now acquired are probably unparalleled in the world's development of physical undertakings of magnitude.

The erection of simple structures considers chiefly girders, roof trusses and columns. Girders vary from the dimensions of rolled I beams to those of solid plate girders more than 120 ft. long and weighing over 100,000 lbs. each, or of lattice girders of 150 ft. or more in length, the girders up to 120 ft. long having been shipped from the shops in single rigid pieces. Such long and heavy pieces must be loaded skillfully to ride the railroad curves, and each requires from three to five flat-cars

for its transportation. The girder is supported at each end on a transverse beam that has an iron bar or old rail on top on which the girder rests, and can easily slip to conform to the chords of the curves. This transverse beam is supported by the centres of two or more longitudinal ones whose ends rest on transverse beams placed on the car floor and thus distributing the load on two lines, one at each end of the car, or else rest on another set of longitudinals that are set on four transverse beams, one of which would thus be directly over each axle and sustain one-eighth of the total load, half of which is carried by each end car, the intermediate ones acting only as spacers. When there is sufficient head room girders may be loaded edgewise, but otherwise and more often they are loaded flatwise. Whenever practicable, they are not unloaded until brought across the openings they are intended to span and parallel to their final positions from which they do not vary longitudinally and not more than is necessary transversely. They are then usually raised a little by hydraulic jacks and supported by timber blocking till the cars are run out from beneath them and then jacked down and skidded to their seats, or, less frequently, are lifted from gallows frames and turned if necessary and lowered by tackle. These gallows frames, one at each end, ordinarily consist of single bents of, say, 12 x 12 posts and single or reinforced caps that just span one or two tracks and are guyed both ways. When no old or temporary track exists across the opening, the girders have been unloaded at one end of it and placed in the required position by protrusion, *i. e.*, pushed out cantilever-wise over a stationary roller on the abutment until the forward end reached its seat on the opposite side. This method requires either a pilot extension, a rear counterweight, overhead guys or intermediate supporting rollers; after it is half way across a pilot extension would generally be used and would be a long beam lashed firmly to the girder so as to engage a roller on the further side before the center of gravity of the girder passed the first abutment.

A remarkable example of this method of erection is that of the Souleuvre Viaduct in France; its spans of riveted lattice girder type, were completely assembled about 1600 ft. from one abutment, connected together as continuous spans, and rolled out on fixed rollers; the spans weighed nearly 100,000 lbs. each and had in front a trussed pilot 66 ft. long that weighed 40,000 lbs. The bridge was protended by the revolution of the fixed rollers at each pier. These rollers were turned by ratchets operated by long levers, one on each side of the span, connected by a cross-bar over the top of the bridge. Men walking back and forth on the top of the bridge pushed this cross-bar before

them and thus turned the rollers, but considerable difficulty was experienced in securing uniformity of action between different gangs. This example is notable in that it should have been successful, and for its striking difference from American practice.

Roof trusses up to about 100 ft. span are generally lifted and set in place as one complete finished truss, whether with rigid or flexible joints. If with riveted joints, they have been shipped from the shops in one, two, three or four sections each, that are riveted together on the ground at the site; or, if pin connected, they are assembled there, and in either case raised and set by a gin-pole or derrick that moves backward with each successive truss.

These trusses often depend largely upon the roof sheathing boards for lateral bracing, without which they have little transverse stiffness. They are also likely to be set on slender isolated columns, and require special care in guying until permanently braced after being released from the derricks. When supported on columns the trusses may be assembled between them exactly parallel, and each with its lower chord nearly in the vertical plane of its final position, but if supported on masonry walls they must be assembled with their lower chords sufficiently oblique to clear and be adjusted after rising above the tops of the walls. They may also be assembled on the same platform or blocking at one end of the building and raised there without moving the derrick, and skidded along on top of the walls to their respective positions; but this method will usually be more difficult, tedious and hazardous than moving the derrick to raise them in position. This method was employed at one of the mills in the famous Homestead Plant, and either one end of a certain truss was advanced beyond the other, or else the flexibility was so great that it was pulled out of a plane and the lower chord became curved horizontally so that it fell off and tumbled to the ground.

Two gin-poles are often used together to raise a roof truss, each gripping it about one-fourth or one-fifth of its length from the centre, and of course, always above its center of gravity. The speaker once used this method of erecting a rolling mill roof of over 140 ft. spans whose very light trusses had slender gas pipe struts and deck beam top chords, and were about limber enough to be be considered funicular machines, but were handled without difficulty by the renforcement of planks judiciously lashed on.

A gin-pole is simply a timber mast with four guys and a sheave at the top over which the hoist line leads to a crab bolted three or four feet from the bottom. In use it should always be inclined a little from the

vertical so that it overhangs its burden and gives a positive strain on the back guys and on them only, the front guys coming into service when the pole is moved. By taking up or slacking the guys the truss may be very quickly swung backwards, forwards, or transversely and adjusted to position, and for heavy work a tackle is advantageously used to operate at least the back guys.

The foot of a gin-pole is generally supported by and shifted upon a plank or timber along which it is pinched with bars or pushed upon rollers. Gin poles are ordinarily from 40 to 60 ft. long and up to 16 inches square at the butt. In erecting a lofty dome recently Horace E. Horton, of the Chicago Bridge Works, used a trussed gin-pole 120 ft. long. Gin-poles are often rigged with ¾ inch wire guys and 1½ inch manilla line that would, according to the load, be operated directly by the crab or be rove over a fixed and loose single block or a two-three pair of blocks.

An A derrick, is two inclined masts braced together and united at the top; needs but one guy and is very often preferable to a gin-pole.

A gin-pole must always be carefully handled and may be easily raised or lowered by a boom or an A derrick that is likely to be found at any large building, provided the height of the pole is not more than twice that of the derrick, which can then pick it up just above its center of gravity, and swing it into its vertical position. If the gin pole is only a little too long for this it can still be handled, as by counterweighing the butt. A short pole can be raised by fastening the foot and blocking the other up beyond the center of gravity until the angle is great enough to enable it to be revolved by a rope leading to the ground, but much the easiest and best way is to provide a secure resistance for the foot, making a virtual hinge there, and hoist it from the ground by a line led over the top of a shears, which need only be any convenient pair of timbers lashed together at the top. When the pole rises high enough to carry the rope off from the shears they will be no longer needed, and if the hoisting crab has been properly set it will continuously pull the pole up to its vertical position. The foot of the pole must be carefully watched, and reliable men stationed at the guys which always, whether moving or raising the pole, should be kept free from slack.

When the span is very great, or when the ground underneath must be preserved free from obstruction, the roof trusses are either assembled and erected from a strident traveller, or a tower, or upon a moveable platform whose surface conforms to its lower chord or intrados, is as long as the span and usually is a little wider than the distance from out

to out of the two most distant adjacent trusses, so that each pair may be simultaneously erected in position, braced together, and left in stability while it moves forward two panels lengths to the next pair.

Lofty viaducts have been erected with the utmost simplicity by booms carried on the structure itself as its towers were built up section by section from the ground.

The famous Kinzua viaduct, over 300 feet high, was erected in this manner several years ago, but the method is evidently best adapted to locations where the iron work is most readily delivered in the bottom of the chasm, which is not usually the case, and probably would not now be used except for very short structures or where the spans between towers were extremely long, the material being chiefly handled from overhead in the present practice.

By far the most usual and generally economical way of erecting viaducts, including the Metropolitan Elevated Railroad Structures, is by means of an overhanging derrick that moves on top of the completed portion and reaches far enough beyond it to set all members in the next one or two panels in advance of its own support, setting and maintaining each piece until it is braced and self-supporting; the connections usually being quickly and temporarily bolted up to enable the derrick to move forward onto the new panel and commence erecting the next one before the main joints are drifted and riveted and secondary connections completed. These derricks are called "Erecting Travellers," and comprise essentially a base moving on the finished work, and carrying the hoisting engine, coal, &c., that partly counterbalance the overhang and its burden. A reach of 60 feet usually suffices for elevated railroads whose travellers may consist simply of long, single beams, mounted on central wheels and set with the front end slightly elevated and the rear or trailing end lashed or otherwise secured to the longitudinal girders as was done on some of the earlier Brooklyn work. Generally, however, a braced platform of the full width of the structure, carries a vertical head frame in which are set masts of two or three boom derricks, whose booms are usually trussed and swing nearly around a semi-circle so as to be able to pick up the iron from the side of the street and swing it into position. Often these booms are arranged so that the two side ones can set and hold the columns of the next panel while the longer centre boom puts the transverse girder in position upon them and after it is connected to them maintains the whole bent until the released side booms set the longitudinal girders and make the whole panel stable.

Railroad viaducts are generally considered to be most economically

proportioned when the towers are 30 ft. wide and 60 ft. apart, and many have been built approximately to these dimensions, thus requiring an overhang to support a transverse beam or pair of column sections at a distance of 90 ft., a short longitudinal girder at 75 ft. and a long one at 30 ft. These travellers always consist of two parallel trusses, usually combination, always firmly braced together horizontally and anchored to the structure when in service. Sometimes the overhangs are single heavy beams with iron guys from the end and intermediate points to the top of a mast placed over the end of the supported part, and guyed back to the rear of the platform. But they are more often square Howe or Pratt trusses not unlike bridge spans, overhanging about half their length, and having cross-beams and eye bolts in the lower chords from which to suspend the tackles required to lift and support the pieces in the different positions of the towers and spans.

Viaduct travellers are designed according to special conditions so as to receive the members for erection directly underneath the overhang, or from cars that run on its own track level and come from behind up to or underneath the main platform and deliver to trolleys that carry the hoisting tackle out on the overhang, or to booms that swing it around, or to falls that slack it off to position. In building the St. Paul High Bridge across the Mississippi River, Horace E. Horton erected the long and lofty viaduct by a huge travelling tower that was 150 ft. high by 68 ft. square, and straddled the 125 ft. high trestle bents, with a clear spring of more than 135 ft. high. It was built chiefly of 5 x 10″ and smaller sizes of timber with iron main tension diagonals; ran on eight double-flanged wheels, and was probably the largest and tallest traveller ever constructed.

As crane bracing and horizontal trusses must be permanently supported by columns, walls or vertical trusses, the supports greatly facilitate their convenient erection, which is almost invariably accomplished by simple tackle directly supported and operated by stationary crabs or engines.

The only primary structures remaining to be considered are Long Span Bridges, *i. e.*, say above 150 ft. long. They may have either pin or riveted connections, or be suspension bridges or arches. Most of them in this country are the former, although there is a growing tendency to design arches for locations where the geological formation affords good seats and in receiving the thrust saves the metal required for a tension chord.

Cantilevers and Suspension bridges are the only types that are self-supporting during erection. The former may include drawbridges when

they are erected symmetrically with the panels simultaneously added each side of the centre so as to balance each other upon the pivot pier, but they are generally erected upon the fender piling in the axis of the river each side of the pier.

In cantilevers the anchor arm is first built on false work and counterweighted so as to enable the channel arm to be built as an overhang, its members being self-sustaining as soon as each panel is connected.

American cantilevers are almost invariably connected by a suspended centre span of from $\frac{1}{3}$ to $\frac{1}{6}$ of the total opening, and this is usually erected as an extension of the cantilever arms from each side, special temporary or permanent stock being provided in the truss members if necessary to meet the erection strains, which are usually allowed a high unit value.

Wire Suspension Bridges are commonly erected from their own cables, which, when twisted rope is used, are drawn across the river in strands and then lifted to the tops of the towers.

The large cables are merely bundles of parallel straight wires that are carried across singly by special machines, looped over the end pins, and spliced at the end of each coil so as to form one practically continuous filament, the different individual catenaries of which must be carefully adjusted and secured to uniform tension and then compacted and encased. After the cables are completed the members of the floor and stiffening trusses and working derricks and platforms are supported readily from them.

Arches are generally assembled on false work, but may be sustained without it by commencing at the skew backs and supporting each section by overhead guys, as was notably done in Ead's St. Louis Bridge. Such a method, or that of temperary reinforcements to enable an ordinary truss to be erected cantilever-wise, may be termed artificial self-support.

Tubular Bridges are happily obsolete in this country, the only important one in America being the Victoria Bridge across the St. Lawrence at Montreal. It has many long spans,* over deep and rapid water, the bottom is too rocky for pile driving, and I cannot cite positive statements but have been informed that the superstructure was built in situ in the winter, upon false work erected on the ice which forms and packs and freezes there to remarkable thicknesses so that higher up on the river it is not uncommon to run ordinary locomotives

* As nearly as I can recollect about 17 spans of about 250 feet each, and one or two of 300 feet.

across on it. Stevenson's other great tubular bridges at Conway and the Straits of Menai, England, were built at the water's edge, floated complete to the unfinished pier, and raised many feet to final position by hydraulic jacks, their masonry seats being continually built up close under them as they rose.

With the above exceptions, long span arches and trusses are supported on false work substructures until self sustaining.

Fixed False Work.

This may consist of a simple temporary suspension bridge with a more or less stiffened floor for comparatively light work or where the height or cost of false work would be prohibitory, or where it could not be safely maintained. Excellent examples are afforded by the South American work of L. L. Buck who erected some of the first railroad bridges in the Andes mountains, with the simplest equipment and unskilled labor; the structures spanned deep chasms traversed by mountain torrents that were liable to sudden floods that would destroy trestle false work and the trusses could not be erected cantilever-wise. A suspension bridge was therefore made across the river, and on its floor the trusses were erected by a light special derrick that commenced at the center and erected to one end by balanced overhangs that were lowered to allow it to return inside the erected structure to the center and thence erect the remainder of the truss.

In an adjacent viaduct crossing, a cable was stretched from side to side, and upon it a trolley hoist travelled by gravity, receiving the iron members on top of the bank and lowering them as it carried them out to position so that the resultant motion carried them in a nearly straight line to their required locations in the towers.

Reverse conditions existed at the Elkhorn viaduct erected recently by Coffrode & Evans, Philadelphia. The timbers for a 600 ft. permanent viaduct 140 ft. high were delivered, framed and connected up in sections of trestle bents, at the centre of the bottom of the chasm where a stationary hoisting engine raised them, traversed them along the line of the structure and set them in position by means of a trolley travelling on a suspension cable and operated by hoist and traverse lines leading from it to snatch blocks, at the foot of the opposite towers.

Trussed span false work has been used chiefly where there was great danger from floods, drift or scour, or where it was imperative to offer the least obstruction to navigation.

Howe truss or combination trusses have generally been used; and can be quickly erected on trestles to seat on the bridge piers and become self

sustaining, and permit the removal of the trestles and allow plenty of time for the removal of old and assembling of new structures.

The long spans of one of the first Missouri River Bridges were erected on one temporary Howe truss deck span that was assembled on trestle work of the required height, then transferred to two wooden towers built on the decks of pontoons which were floated with their burden to position between the piers; water was then admitted to the pontoons and they sunk sufficiently to deposit the span on the piers and clear it and be removed, leaving a platform safe from floods, for the assembling and connecting at leisure of the permanent span. After it was swung the pontoons and towers were brought back underneath, pumped out till their bouyancy lifted the Howe span which was then transferred to the next opening and seated as before, for the erection of another span and so on.

This erection was notable for the length of the wooden span, the height of the towers, the swiftness of the current and the success and economy of the operation.

Several years after this erection was that of the bridge of the Canada Atlantic R. R. across the St. Lawrence River at the head of the Coteau rapids, with a 355 ft. swing span and one 139 ft. two 175 ft. four 223 ft. and ten 217 ft. fixe spans, The water was from 20 to 30 feet deep, with a current of 5 to 7 miles an hour, making it impossible to erect false work on the rocky bottom, and very difficult to build the masonry piers which were constructed in bottomless wooden caissons that were floated into position, loaded and sunk with inside canvass bottom flaps on which the concrete was laid.

Three miles above the site in a sheltered bay, false work was built parallel to the shore and at its ends transverse tracks were built out to deeper water, Between these tracks and parallel to the false work were moored a pair of timber pontoons each 90 x 26 x 6 ft. deep, and braced together 70 ft. apart. On the deck of each pontoon a trestle a little higher than the bridge piers was built. The permanent spans were assembled and connected complete on the false work; skidded down the tranverse tracks above the towers which rose and lifted them free when the water was pumped out of the pontoons, and supporting them at two panels distance from each end were slacked off down the river and sunk between piers enough to deposit the span on its seats after which they returned for another span and so on.

In the erection of the Harvard Bridge at Boston, the long plate girder spans were built on shore, lifted by a traveller that overreached their ends and at high tide carried out by it deposited on a special pontoon,

that was towed to position and with the falling tide deposited the girders on their pier seats.

The Brunot's Island Bridge near Pittsburg, and the Hawkesbury Bridge in Australia erected by American engineers, are notable illustrations of moving very large spans at a great height upon pontoons.

TRESTLES.—These are essentially sets of columns in vertical planes transverse to the axis of the structure, they are in rows or bents of four or more with transverse and longitudinal bracing, the latter either continuous or connecting alternate pairs so as to form towers. When the height is considerable the trestles are built in stories so as to give convenient sections. The simplest framed trestling has bents each composed of two plumb posts and two batter posts, a cap, a sill and two diagonal braces, and the bents are braced longitudinally by a horizontal ledger piece at the top and a diagonal from the top of one to the foot of the next, on each side. Uusually the caps are connected by two or more lines of stringers which should rest directly above the adjacent tops of the plumb and batter posts, the latter having a run of 1 in 12 to 1 in 4 horizontal to vertical according to circumstances. The dimensions of the timber should be proportioned to the load carefully in high or exposed structures or for very heavy burdens, but must never be made very light, for the connections never develop all the efficiency of the sections, and weight and stiffness are generally more important than direct theoretical strength, so that careful judgment and experience are much more reliable and necessary than elaborate calculation of strains.

For ordinary simple trestles 10 x 10 and 12 x 12 posts and caps, 3 x 10 or 3 x 12 braces and 8 x 16 stringers are much used, and for lofty and elaborate structures the variation is more in the number and arrangement of pieces than in their sections, though some 6 x 6 and 8 x 8 are used for secondary braces and larger ones for important members if obtainable. But the few sizes mentioned with 2 inch platform plank and plenty of $\frac{3}{4}$ and $\frac{7}{8}$ bolts, large washers, wood screws and steel spikes comprise most of the bill of materials except occasionally screw ended iron tension rods with cast angle blocks. Mortise and tenon joints are seldom used; sometimes a batter post is toed in, but most joints are butted and covered with a splice or "batten" piece each side; usually a 2 or 3 inch plank as wide as the timber, and from 4 to 6 ft. long, secured with four or more bolts.

Sometimes double caps are used, one piece on each side of the posts, but usually single pieces of the same thickness as the posts are put in their vertical plane, and between the tops of the lower and the bottoms

of the upper sections, and all are bound together by batten pieces across the caps.

Trestle bents may be from five to twenty feet apart and of any height from 4 to 150 feet. Smaller heights are blocked up solid and greater ones are usually avoided or otherwise provided for. Stories do not ordinarily exceed 20 ft. in height. Ledger pieces and diagonals are sometimes spiked on, especially to promote rapidity of erection, but the latter at least should generally be bolted eventually.

When framed trestles are to be set with sills on a river bottom, careful soundings must first be taken, preferably with a pole, and a profile made to determine the heights of the posts. The bents are laid out with uniform tops and bottom widths varying to suit the different heights and bottoms. They are bolted together on shore, floated to position, swung up and set vertically from a derrick or pile driver boom or from the cap of the last bent if it is high enough, and stayed by ledgers, braces or stringers.

If the water is deep the lower end of the longitudinal diagonal brace is bolted to the foot of the batten post before the bent is set, so as to easily bring it eventually at the desired position under water.

Sometimes the bottom is explored ahead, or the bents raised from a "balance beam," which is simply a long heavy timber projecting half its length beyond the last trestle set, upon whose cap its middle is supported while its rear end is under the cap of the preceding bent.

Ordinarily the "mud sill" is a sufficient footing when framed trestles are admissible, but sometimes special provisions are requisite on account of severe burdens or very soft bottom. In trestling over a very soft mud John Devin secured very cheap and efficient footings by spiking old railroad ties transversely to the bottom of the sills of his trestles, and thus constructing what was substantially a grillage under each bent.

Pile Trestles.—Wherever piles can penetrate they form the best foundations for false work. They are driven to a moderate refusal and loaded much more heavily than in permanent structures. They are used in single lengths up to about 60 ft. beyond which they must usually be spliced, the joint being generally a square butt with comparatively narrow batten splice pieces bolted or spiked on all around it. At the Poughkeepsie Bridge piles were thus driven 120 feet through the the mud and water.

Piles are driven either by a floating driver or by one running along on top of the completed bents. The batten posts are driven by inclining the ram guides to correspond to the required inclination.

If the top of the trestle is within ten or fifteen feet of the surface

of the water the piles are usually sawed off and capped just below that height, but if the elevation is much greater they are usually capped near the water level, and one or more stories of framed trestle built upon them.

Sometimes, to prevent brooming, piles are driven with a temporary iron ring or ferrule on top; sometimes they have iron points or cast shoes at the bottom.

Great judgment is required in driving piles which should be controlled far more by skilled experience than by theoretical considerations; sometimes, when the penetration is excessive, if they are allowed to rest a few hours they will resist heavier blows of the ram than previously drove them, and be amply safe for severe static loads. At other times the pile will refuse to penetrate and will bound out at every blow, but will sink steadily if loaded with a dead weight and tapped or twisted.

Some remarkable and apparently extravagant pile work was done at the Gour-Noir R. R. viaduct over the La Vézère River, France, where the 213 ft. main stone arch span 26 ft. wide between parapets and 5′ 7″ thick at the crown, was built on centering carried by seven continuous wooden trusses 14′ 6″ deep that each rested on sand boxes on the tops of oak piles 12 inches or more in diameter; their bottom ends cut square and shod with plate iron. The river is subject to floods and has a granite bed into which holes 3 feet deep were drilled and the piles wedged and cemented in them. Quartz veins were encountered in which only one tenth of an inch per hour could be drilled until coffer dams were built. The cost of setting 126 piles was about $6,800.

Timber dimensions for Eastern trestling seldom exceed 12 and 16 or at the utmost 20 inches, but a recent letter from a bridge man on the Pacific coast says his regular bills call for sizes like 11″ x 30″ x 42′, 20″ x 20″ x 48′ and 7′ x 18″ x 64′, while he saw some logs being sawed into timber 48″ x 48″ x 50′ and 20″ x 20″ x 100′.

Sometimes, to afford passageway for navigation, a few bents will be omitted at the bottom of a line of false work, and the opening will be bridged by a high temporary span to carry the continuous top bents. Such an opening may be as much as 75 ft. wide at the bottom and narrowed at the top by inclined posts so as to permit a sort of queen post span of comparatively short pieces or the use of stringers borrowed from the permanent structure to close the opening at the top.

In the early days of iron bridge building, it was frequently customary to build "two story false work," *i. e.* to provide two complete platforms the whole length of the bridge, to support top and bottom chords throughout, but now it is almost universal to support only the bottom

chord directly, and to make the rest of the truss self supporting when assembled upon it, stability being of course assured as soon as any two opposite panels of the two trusses are assembled and connected transversely to each other.

Wooden derricks or towers usually traverse the false work to lift and place various members or are fitted to follow out at the extremity of a cantilever arm and overhang it so as to assemble and connect the successive panels upon which it continually advances. These structures are called travellers and properly belong to the

Working Plant,

and form the most important item of erection equipment. A traveller in its simplest form consists of one transverse bent double guyed or two bents braced together longitudinally to form a tower, each bent consisting of a post each side and a cap and knee braces at the top, commonly however, there are three or four bents on large work, leaving at each side a vertical post and a batter post braced together, the latter flaring out at the top and their feet united on a heavy double longitudinal stringer, underneath which are three or more double flanged wheels that run on special tracks laid on the ends of the trestle caps, the caps of the traveller bents are developed into Howe or Warren girder trusses, and usually carry longitudinal straining beams from which the hoisting tackle is hung. The bents are braced together with iron or wooden diagonals, and the structure carries working platforms and hoisting engine and perhaps other machinery, besides sometimes tracks for material cars. Generally the traveller goes astride the completed structure and clears every portion of it, and is so required to have a large free opening from end to end with no transverse connection between the bottoms of the posts; but it sometimes is designed to go inside the trusses, or backwards in advance of them from a starting point, or upon the top chords, in which cases it may have bottom and interior cross and diagonal bracing.

Traveller Bents, as at Poughkeepsie, Wheeling, Memphis &c., have been made more than 100 ft. high, assembled and connected in a horizontal plane and the first one erected by simply lashing down the heel and revolving it into a vertical plane by two or three carefully adjusted lines on each side, attached at different points, and leading over shear poles to the hoisting engine. Of course the lofty timber frame was securely held by guys front and back that were kept constantly taut. After the first bent is erected the second one is easily hoisted from it, both are braced together and the structure immediately becomes stable and rigid.

In erecting an ordinary simple truss span the members of the floor system and lower chords are distributed on the false work in position before the traveller lifts and assembles the other members, commencing either at one end and going straight across or beginning at the centre and working to one end and then returning and working out to the other end.

The trusses are carefully placed in alignment and vertical plane, but are blocked up at the joints, where they rest on wedges, to a much greater camber curve than exists in the completed structure, so as to allow the adjustments for the final connections being made by driving out the wedges.

In suspended spans between cantilevers, the end piers rest in expansion slots in the cantilever arms, thus permitting longitudinal motion that would allow the portions of the centre trusses to hang down and make the final connection impossible unless special provision is made for adjusting their length and controlling their position and inclination. This usually consists of, at each of the eight slots, a fixed and movable roller separated by a wedge which is commanded by a powerful screw so that by entering or withdrawing one or both of its wedges the extremity of any truss segment may be raised, lowered, protruded or withdrawn.

Usually they are set in the beginning so as to allow for the deflection of dead load and traveller, and still be inclined above the required cambered position so that the adjustment requires only the slacking off of the wedges.

Before this device was well known the speaker was required to provide erection adjustment for the second large cantilever bridge ever built in this country, at St. Johns, N. B., and designed simple stirrup irons or U bolts that engaged the movable top chord pins and had screw ends passing through fixed bearing plates against which their nuts rested. This afforded a tension adjustment for drawing or letting out the top chord, while large bolts screwed through solid boxes in the ends of the lower chords any having rounded ends bearing in hemispherical cups in a casting bolted to the end of the cantilever formed a compression adjustment and could be easily set up or slacked off so as to lengthen or shorten the lower chord. This method proved simple, economical and satisfactory.

On the great cantilever of the famous Memphis Bridge, erection adjustments were provided for *only one end* of the suspended centre span, and they proved entirely insufficient to enable the final connections to be made, the opening between the ends of the lower chord sections being about 4 inches too short to admit the piece of required length; the incomplete erection was brought to an abrupt stop and the situation was

critical, but the danger was averted and the problem ably solved by William Baird, contractor for erection who inserted an ingenious and powerful toggle joint at the centre of the span by which he shortened the lower chord thus increasing the camber enough to release the packing that fixed the pins in the unadjustable slots. The packing was removed and the toggle released, reducing the camber until the lower chord opening admitted the middle section and the final connection was made, thus accomplishing one of the most brilliant achievements of erection work.

Working Plant comprises travellers, derricks, hoisting engines, tackle, pile drivers, pumps, locomotives, cars, differential hoists, hydraulic and screw jacks, dynamometers, steam, pneumatic, hydraulic and electric appliances and hand tools. Stationary derricks are much used for unloading and handling material and for hoisting secondary members. They are ordinarily of the familiar mast and boom pattern, with hollow vertical mast pivot through which the fall lines lead to the hoisting drums, and are generally rigged with manilla running gear and wire rope standing guys. Sometimes they are stiff-legged, and sometimes have also bases of timber sills enabling them to be easily moved. Balanced derricks are sometimes used. At the Washington Bridge across the Harlem River all the material for one of the 510 ft. arches was lifted to a distributing platform by a balanced derrick that had twin overhangs, a bearing on top and a friction collar and rollers at the bottom of the trussed arms. Small four-wheeled trucks or "lauries" are usually used for distributing the iron work on the span and bringing it from the yards, but these often are provided with special lifting devices for expediting loading and unloading, especially if there is no yard derrick. A car having two braced vertical posts in the middle, capped by an overhanging beam with a tackle suspended from each end is very convenient for picking up a pair of stringers and bringing them and unloading, one on each side of the track, nearly in the required position. A very effective arrangement was devised by a young erector, that simply consisted of a long trussed beam with its forward end somewhat elevated and carrying a tackle that was mounted close to the front end on a small car; it could be run out in the yard and the fall belayed and the lever tipped down to hook on to a heavy piece; then several men, mounting the long arm of the lever, would raise the load and push the car to any required place, when its burden would be lowered by slacking off the fall line. This device was especially convenient for handling floor beams.

The many varieties of hoisting apparatus in use are generally some-

form of multiple spooled engine, with capstain heads and drums; some of them have eight spools, each driven by independent gearing and commanded by clutches and brakes, so that any one may haul up or slack off independently, the rope only taking two or three turns around the spool and then being tailed off so as to maintain the same efficiency always. These engines usually have a locomotive gearing to enable them to propel themselves on standard gauge track. An ingenious method has been employed to hoist them to the top of high false work. At the Poughkeepsie bridge a six-spool engine was delivered by a boat underneath the traveller, from whose top beams, 250 ft above, four sets of tackle were hung and their lower blocks were hooked on to the engine bed, one at each corner; the fall line of each was wrapped around a capstain head and kept taut by a man mounted on the frame. Another man started the engine, and as the fall lines were wound up and tailed off, the engine pulled itself and its five men up to the top of the false work and was let down on beams slipped under to receive it. Sometimes a locomotive can be advantageously employed to hoist heavy members, as at the Niagara R. R. Suspension Bridge, where the heavy saddles were hoisted to the tops of the 80 ft. towers by a line rove through upper and lower snatch blocks, and led from the latter to a locomotive that simply steamed off with it and drew the load up after it. The running tackle used should be best manilla rope; 1¼, 1½ and 1¾ are the common sizes, generally rove through double and treble blocks with lignum vite or steel shelves 16″, 18″ or 20″ in diameter, and steel hooks and cases. Members are generally lifted with chain slings which must be properly fastened to avoid cross-straining the links, which can be very easily snapped. Special hook clamps are often provided to fit the flanges of girders. For extra heavy strains a luff tackle may be used, and consists merely of a second purchase attached to the fall line of the first. When this is commanded by a heavily geared windlass or a hoisting engine great power is developed, but is slow and troublesome in its application.

Very heavy pieces and large masses or assembled structures may be moved horizontally on greased skids by hydraulic jacks that are made of from 5 to 100 tons capacity, and can also raise or lower it, but should then be closely followed by solid blocking.

In commencing to drive a connecting pin, the holes in the different members often do not match, and a square-ended pin could not be entered. Therefore, the end which is shouldered and threaded for a nut receives a pilot of the same diameter as the body of the pin, up to which it is screwed to fit tightly, while the front end is made conical

and can enter a half-hole into which it is driven, drawing the pieces into position and allowing the pin to follow easily. The pin should always be driven by a wooden maul or ram to avoid battering its threads. An iron-hooped beam or log suspended by its center of gravity at the pin, held from a considerable height and swung against the pin, does excellent service.

The specifications for most large recent bridges have required machine field riveting which, in this country, has been done by pneumatic or hydraulic tools similar to those used in the shops. At the Memphis Bridge the riveters for the floor system were simply swung by long ropes; at the Poughkeepsie Bridge they were swung from a trolley that ran on a longitudinal track, moving in a transverse arc and all revolving about the center of a small traveller that cleared the inside braces of the main traveller. At the Washington Bridge the arch rib splices were riveted up by a machine that hung from two differential hoists carried by a trolley whose deeply grooved wheels rolled on 4″ round bars that themselves rolled freely on the horizontal braces. Electric field riveters are being much used abroad. A recent pattern has a small motor that by reducing gear operates a screw piston which develops hydraulic pressure for driving each rivet.

Adjustments of tension members sometimes are required to be made accurately and verified, this can be done by interposing an ordinary spring dynamometer so as to form a temporary link in the connection. But this is many times a needless refinement, since the proportionate, if not the actual, tensions of members can generally be closely estimated after some experience by striking them with a hammer. In adjusting over 600 floor beam suspenders of the Niagara Bridge the speaker was able to estimate the strain by feeling of the ropes almost within the limits of graduation of the dynamometer.

In the same bridge L. L. Buck reinforced the original anchor chains by additional new links, and accurately adjusted the load taken up by the latter by the elongation produced.

When only a slight discrepancy at first exists between tension bents of the same members they will usually adjust themselves by proportionate elongations, but if the variation is too great it may be sometimes eliminated by heating both bars (as by wrapping them with oily waste and igniting it), and allowing them to set themselves in cooling. In the erection of the steel arch spans of the Ead's Bridge at St. Louis, the expansion of the metal in the heated atmosphere made it impossible to insert the last segment of one of the arches until the whole arch had been cooled by ice packed around it in a trough.

Thus it is seen that in bridge erection unforeseen and perplexing contingencies continually arise, and must be met by all the resources of science and mechanics.

The general equipment of a complete general erection outfit should comprise full kits of carpenters' and blacksmiths' and masons' tools, portable forges (these may be improvised with half barrels filled with clay, and a large bellows pumping into another large barrel with tuyer pipes to three or four of them will furnish very satisfactory and economical blast, and may be very convenient for such work as riveting up buckle plate flooring), riveters' outfits, tool steel, plenty of steel spikes, fitting bolts, long bolts and washers, hand screws, hand chisels and gouges, ratchets, reamers, screw and set wrenches, large key wrenches with rings, pinch bars, crow bars, hand hammers, sledges, a 50 ft. and a 100 ft. steel tape, rubber clothing, lights, and the more important tools, &c. mentioned above.

American bridge engineers design their structures with careful consideration for the erection requirements, and of the combination bridges lately built in the far west, some, if not many have been specially constructed to afford facility for replacing the wooden compression members with iron without disconnecting them or impairing the integrity of the structures.

A large and increasing proportion of the bridge erection here to-day consists in the renewal of existing structures where it is almost invariable demanded that the traffic shall not be interrupted. When permissible the road is usually moved to a temporary crossing on one side of the old structure which is then demolished and replaced unrestrictedly, but this method is not often practicable and numerous other expedients are resorted to, most often, probably, trestle work is put up under the span and the track transferred to it, as well as the old structure after it is disconnected. As soon as it is removed the new structure is assembled upon the trestles, everything being scrupulously made to clear the trains or only used in fixed intervals between them. Sometimes the old bridge is made to support the new one till the latter is swung and self-supporting, when it in turn supports the old one until completely removed. Sometimes the old structure is taken out piecemeal and replaced by the new, or clamped to it for certain periods.

Some very remarkable achievements have been accomplished by L. L. Buck, whose work on the R. R. Suspension Bridge at Niagara Falls is a masterpiece of skill and ability. He, at first, opened the anchor pits, disconnected the main cables, replaced numerous corroded wires, removed the anchors, replaced them and added new links and pins to

their chains. Afterwards he replaced the whole suspended wooden superstructure with steel and iron floors and trusses. At another time he removed portions of the masonry towers, and put in new stones, and finally he replaced the massive towers with steel structures standing on substantially the same foundations, and accomplished all the difficult operations quickly, cheaply, and without loss of life or any serious interruption of traffic.

The rapidity with which erection work can be executed is illustrated by the bringing of the material for a 200 ft. railroad bridge from a storage yard 1000 ft. away, and erecting it in 16 working hours after false work was ready.

The 518 ft. span of the Cairo Bridge was erected by Baird Brothers in 6 days; two spans were erected and the false-work and traveller twice put up and moved in one month and three days, inclusive of five days of idle time.

Bridge erection is subject to many dangers and serious accidents appear to be inevitable. Those occurring to single individuals are often not the fault of the victim, who is frequently injured by an article dropped by someone else or knocked off from the work by some carelessness. An experienced bridgeman seldom falls from a great height through dizziness or missteps, but may do so by carelessly stepping on a loose plank. Some terrible accidents have occurred by the collapse of false-work, trestling, and travellers, some of them perhaps due to derailments or breakages while hoisting heavy pieces, or possibly to outright general weakness, but comparatively few are ever exactly determined, except where, as is too often the case, trestles are destroyed by floods scouring out the bottom underneath them or piling vast quantities of drift against them, as was the case at Wheeling, W. Va., and has been frequent in the Ohio river bridges.

Although Cornell University is still young, many of her sons are making successful bridge engineers and erectors. Theirs is a laborious, exacting calling, but exceeded by none other in the magnitude of its undertakings, and the courage and resourcefulness demanded. Technical skill and professional training are valuable advantages for this work, but among its ablest leaders are practical men with no mathematical or scientific training or academic education, to whose skill and wisdom and genius the development of this great branch of engineering is largely due, and I call those men as truly *engineers* as if they held diplomas from every college in the land.

No other calling demands and receives the experience, courage, good judgment and personal endurance displayed by the leaders and skilled

workmen in Bridge erection. They must construct the loftiest and most difficult scaffolding, solely by their own resources, often in remote and dangerous positions, and upon them must handle and perfectly adjust heavy girders and huge chords, etc., weighing perhaps 100,000 lbs. while subject to constant peril of destruction by storm and flood, or they must build great trusses in the very path of frequent express trains without impeding their progress or prejudicing their safety. Under such trying circumstances their work is accomplished with a rapidity and accuracy exceeding that in some comfortable and well equipped shops and mechanical plants, and the great address and faithfulness, general integrity and reliability that they exhibit in their difficult tasks, brings them into deserved prominence among constructive workmen. These men are characteristic of our grand nation. Keeping pace with the unparalleled creations of this generation of bridge designers, they have applied no ordinary engineering skill to the devising and execution of erection methods whose success is attested by scores of monumental constructions, and the absence of many great disasters.

The speaker gave a complete list of the general tools required and recently ordered by one of the leading bridge companies for the erection of a four span bridge of over 5000 tons weight, and illustrated all the leading points mentioned, and characteristic constructions of false work, travellers etc., erection methods, and the erection of numerous noted bridges and large roof spans by typical examples in numerous selections from fifty large scale colored detail drawings and dimensioned diagrams and elevations.

THE EFFECTS OF FREEZING ON CEMENT-MORTARS.

Some results from a thesis, by T. W. HILL, '93.

In these experiments one-half of all the briquetts made were put out-doors at once and frozen for 24 hours, both with and without salt. The frozen ones were brought in at the end of 24 hrs., thawed out, and put in the water at the same time as those in the laboratory, and kept there until they were broken. The average of four briquetts in each case, are given in the table.

In the first table the Improved Union cement was used as a 1 to 1 mortar with 30% as much water as cement by weight.

		By weight		
Cement	43.4%		Cement	43.4%
Sand	43.4%		Sand	43.4%
Water	13.2%		Water	12.2%
			Salt	1.0%
	100.0%			100.0%

TABLE I.

Temp. air 66°, water 64°.

No.	Treatment.	Min. T.	1 week.	4 weeks.	13 weeks.
1	24 hours in the laboratory, rem'dr in the water	....	156	216	300
2	Frozen 24 hours, rem'dr in the water	-5°	126	198	277
	WITH SALT.				
3	24 hours in the laboratory, rem'dr in the water	....	118	201	238
4	Frozen 24 hours, rem'dr in the water	2°	104	165	235

Percentages of gain or loss from above.

Comparison of those frozen and not frozen.				Comparison of those with and without salt.			
Nos.	1 week.	4 weeks.	13 weeks.	Nos.	1 week.	4 weeks.	13 weeks.
1–2	-23.8%	-10.6%	-7.6%	1–3	-28.4%	-70.0%	-20.7%
		WITH SALT.				FROZEN.	
3–4	-8.0%	-17.9%	-3.3%	2–4	-18.3%	-14.5%	-15.1%

In the next table the briquetts were made and treated in the same way as those in table I, and with the same percentages.

TABLE II.

Hoffman cement. Temp. air 67°, water 65°.

No.	Treatment.	Min. T.	1 week.	4 weeks.	13 weeks.
1	24 hours in the laboratory, rem'dr in the water	....	55	119	215
2	Frozen 24 hours, rem'dr in the water	2°	57	96	164
	WITH SALT.				
3	24 hours in the laboratory, rem'dr in the water	....	88	136	180
4	Frozen 24 hours, rem'dr in the water	2°	45	110	194

Percentages of gain or loss from above.

Comparison of those frozen and not frozen.				Comparison of those with and without salt.			
Nos.	1 week.	4 weeks.	13 weeks.	Nos.	1 week.	4 weeks.	13 weeks.
1–2	+3.4%	-19.2%	-23.2%	1–3	+37.8%	+14.9%	-11.1%
		WITH SALT.				FROZEN.	
3–4	-48.8%	-19.0%	+7.7%	2–4	-21.0%	+16.6%	+18.2%

In the next table the briquetts are made and treated in the same way as those in tables I. and II., and with the same percentages.

TABLE III.

Milwaukee cement. Temp. air 68°, water 63°.

No.	Treatment.	Min. T.	1 week.	4 weeks.	13 weeks.
1	24 hours in the laboratory, rem'dr in the water	....	54	87	192
2	Frozen 24 hours, rem'dr in the water	0°	42	62	167
	WITH SALT.				
3	24 hours in the laboratory, rem'dr in the water	....	73	140	257
4	Frozen 24 hours, rem'dr in the water	0°	40	68	208

Percentages of gain or loss from above.

Comparison of those frozen and not frozen.				Comparison of those with and without salt.			
Nos.	1 week.	4 weeks.	13 weeks.	Nos.	1 week.	4 weeks.	13 weeks.
1–2	–22.0%	–28.4%	–13.0%	1–3	–35.1%	+61.0%	+33.8%
		WITH SALT.				FROZEN.	
3–4	–45.3%	–51.4%	–19.0%	2–4	–47.0%	+9.6%	+23.9%

In the next table are results obtained from Saylor's American Portland cement treated in the same way as those in the preceding, but with the following percentages:

		By weight		
Cement	44.4%		Cement	44.4%
Sand	44.4%		Sand	44.4%
Water	11.2%		Water	10.2%
	100.0%		Salt	1.0%
				100.0%

8

TABLE IV.

Temp. air 68°, water 65°.

No.	Treatment.	Min. T.	1 week.	4 weeks.	13 weeks.
1	24 hours in the laboratory, rem'dr in the water	....	258	375	423
2	Frozen 24 hours, rem'dr in the water	3°	225	359	447
	WITH SALT.				
3	24 hours in the laboratory, rem'dr in the water	....	235	331	384
4	Frozen 24 hours, rem'dr in the water	3°	203	333	335

Percentages of gain or loss from above.

Comparison of those frozen and not frozen.				Comparison of those with and without salt.			
Nos.	1 week.	4 weeks.	13 weeks.	Nos.	1 week.	4 weeks.	13 weeks.
1–2	-12.8%	-4.5%	+5.6%	1–3	-8.9%	-11.8%	-9.2%
	WITH SALT.				FROZEN.		
3–4	-13.6%	+0.6%	-12.7%	2–4	-9.7%	-7.2%	-24.0%

In table V. are the results from the Hoffman cement when it is mixed out-doors and with the same percentages as used in Table II., and treated in the same way, *i. e.* one half of them were left out to freeze for 24 hours, while the other half were brought in the laboratory as soon as they were made. (Compare with Table II).

TABLE V.

Temp. air 8°, water 42°, cement 40°.

No.	Treatment.	Min. T.	1 week.	4 weeks.	13 weeks.
1	24 hours in the laboratory, rem'dr in the water	....	72	160	270
2	Frozen 24 hours, rem'dr in the water	-3°	58	162	209
	WITH SALT.				
3	24 hours in the laboratory, rem'dr in the water	....	108	157	283
4	Frozen 24 hours, rem'dr in the water	-3°	62	151	220

Percentages of gain or loss from above.

Comparison of those frozen and not frozen.				Comparison of those with and without salt.			
Nos.	1 week.	4 weeks.	13 weeks.	Nos.	1 week.	4 weeks.	13 weeks.
1–2	-19.4%	+1.2%	-22.3%	1–3	+44.4%	-.9%	+13.8%
		WITH SALT.				FROZEN.	
3–4	-39.2%	-3.8%	-5.5%	2–4	+6.7%	-6.7%	+5.2%

In the next table are given the results from the Improved Union cement, mixed out-doors and with 40% as much water as cement.

		By weight		
Cement	41.7%		Cement	41.7%
Sand	41.7%		Sand	41.7%
Water	16.6%		Water	15.6%
			Salt	1.0%
	100.0%			100.0%

(Compare these results with those of Table I).

Table VI.

Temp. air 30°, water 43°, cement 40°.

No.	Treatment.	Min. T.	1 week.	4 weeks.	13 weeks.
1	24 hours in the laboratory, rem'dr in the water	....	76	107	212
2	Frozen 24 hours, rem'dr in the water	6°	57	97	138
	With Salt.				
3	24 hours in the laboratory, rem'dr in the water	....	113	169	149
4	Frozen 24 hours, rem'dr in the water	6°	111	144	191

Percentages of gain or loss from above.

Comparison of those frozen and not frozen.				Comparison of those with and without salt.			
Nos.	1 week.	4 weeks.	13 weeks.	Nos.	1 week.	4 weeks.	13 weeks.
1–2	-25.0%	-9.3%	-34.9%	1–3	+46.7%	+57.9%	-29.7%
	With Salt.				Frozen.		
3–4	-1.7%	-15.4%	+28.1%	2–4	+94.7%	+47.4%	+81.1%

In the next table are the results from mixing Saylor's Portland cement out-doors, and with the same percentages as were used in Table IV. (Compare with Table IV).

TABLE VII.

Temp. air 29°, water 36°, cement 39°.

No.	Treatment.	Min. T.	1 week.	4 weeks.	13 weeks.
1	24 hours in the laboratory, rem'dr in the water	....	256	325	428
2	Frozen 24 hours, rem'dr in the water	6°	226	344	406
	WITH SALT.				
3	24 hours in the laboratory, rem'dr in the water	....	179	278	319
4	Frozen 24 hours, rem'dr in the water	6°	225	323	398

Percentages of gain or loss from above.

Comparison of those frozen and not frozen.				Comparison of those with and without salt.			
Nos.	1 week.	4 weeks.	13 weeks.	Nos.	1 week.	4 weeks.	13 weeks.
1–2	-11.7%	+5.8%	-11.7%	1–3	-30.0%	+14.4%	+25.4%
	WITH SALT.				FROZEN.		
3–4	+25.6%	+16.2%	+25.6%	2–4	-0.4%	-6.7%	-1.9%

PAVING BRICK TESTS.

ABSTRACT FROM THESIS OF D. B. CLARK AND B. N. MOSS, '93.

The advantages of brick as a paving material are fast becoming generally known, with the result that more pavements of this kind are being laid this season than of any other kind. It has been demonstrated that brick *properly placed* in a pavement which is to carry the ordinary traffic of a business street will last longer than asphalt and from one-half to two-thirds as long as granite, and as long as its surface remains in good repair it is unquestionably superior to either. As to price, contracts at present are being let at from $1.10 to $2.00 per square yard depending upon the foundation and availability of the materials. If we leave out of account the disadvantage of tearing up a street occasionally for repairs we find by a simple calculation of the "capitalized cost" that with interest at 6% it is cheaper to pay $1.50 a square yard for brick that will have to be replaced *every ten years*, than to pay $3.00 for granite that will last forever (if such could be obtained).

In making tests for experimental purposes it is customary to include some kind of granite for use as a standard, but we have not done so. The common red brick which was included shows to some extent the severity of the test and brings out quite clearly the fact that *paving* brick are not merely a good quality of *building* brick.

To obtain a sufficient number of samples to be used in our tests, letters were sent to those manufacturers whose addresses could be obtained, asking them to send six samples of their best paving bricks. Eleven varieties were obtained in this way and a twelfth, of common red building brick, was included to give a general idea of the superiority of paving brick as compared with the better known variety.

Six whole bricks were received from most of the parties, and were distributed as follows: Two were used in the test of transverse strength and four were sawed up into approximately two inch cubes. This allowed three cubes for crushing, four for the abrasion test, and three to be used successively in the absorption, freezing and crushing tests.

In the preparation of the cubes the bricks were sawed by a stone-saw using a mixture of chilled shot and sea sand. They sawed slower than granite or marble, it requiring about ten hours to cut through a cross section of one square foot. The roughly sawed cubes were then put on a rubbing bed which consisted of a circular cast iron disc revolving about a vertical axis. To assist the grinding first sea sand was tried but afterward crushed steel was used.

It was the intention on beginning the work to reduce all the samples to two-inch cubes, but the sawing was not done with sufficient exactness and it would have required more time than was available to grind them to exact dimensions, so they were only ground down enough to secure two parallel and equal faces. The dimensions given in the tables show greater variations than should be allowed in comparative tests.

To distinguish the different specimens each brick was given a separate number, that is, beginning at No. I, they were numbered, 1, 2, 3, 4, 5, 6, 7, 8, following with No. II where No. I left off, 9, 10, 11, 12, and so on throughout the twelve varieties, the Roman numerals referring to the manufacturer, the Arabic to the individual brick. When a brick was sawed each cube of the same brick was given a separate letter: 1 A, 1 B, etc., meaning different cubes from the same brick, so that every specimen had a special mark by which it could be recognized throughout the whole series of experiments.

The cubes for use in the absorption test were placed over a nest of steam radiators where a temperature of 150° *F.* was maintained and allowed to remain there a week. They were then carefully weighed with a Fairbank's scale, reading to one thirty-second of an ounce, and immersed in water. After an hour, they were again weighed and the rate of absorption obtained. This was repeated after one day and again after one week immersion. The percentage of absorption is obtained by dividing the increase in weight after immersion by the weight of the cube when perfectly free from moisture. In the table weights are given in ounces and sixteenths of an ounce, and the rate of absorption in percentages. The percentages for the same variety are averaged to give the rate of absorption for that variety.

Absorption and Specific Gravity.

				Percentage of Gain after			Average Percentages.				
No.	Brick.	Letter.	Wt. Dry.	1 H'r.	1 Day.	1 W'k.	1 H'r.	1 Day.	1 W'k.	Sp. Gr.	Average.
	1	A	11— 8	0.54	1.08	1.08				2.421	
I.	4	A	11— 5	0.55	0.35	0.55				2.413	
	4	B	19— 9	0.32	0.64	0.96	0.47	0.76	0.86	2.390	2.408
	10	A	11— 6	0.55	1.10	1.10				2.304	
II.	11	A	10—13	0.58	1.16	1.16				2.307	
	12	A	11—11	0.53	1.06	1.59	0.55	1.11	1.28	2.367	2.326
	17	A	15— 1	1.24	2.48	3.36				2.295	
	18	A	11— 0	1.70	2.27	3.40				2.316	
III.	19	A	16— 5	1.15	2.68	3.06	1.36	2.38	3.27	2.250	2.287
	22	A	11— 2	2.25	3.98	4.50				2.046	
	23	A	11— 0	3.98	5.11	6.25				2.071	
IV.	25	A	10—11	2.34	4.09	4.68	2.86	4.38	5.14	2.085	2.067
	28	A	18— 3	2.75	4.81	5.15				2.093	
	29	A	10—11	2.34	3.51	4.09				2.221	
V.	30	A	16—13	2.60	4.09	4.09	2.56	4.14	4.44	2.187	2.167
	34	A	19— 3	0.98	1.63	2.61				2.178	
	35	A	16—11	0.75	1.12	1.87				2.171	
VI.	36	A	9—12	1.28	2.56	3.20	1.00	1.87	2.56	2.197	2.182
	40	A	11— 1	0.56	0.56	0.56				2.360	
	41	A	7—15	0.00	0.00	0.79				2.352	
VII.	42	A	12— 1	0.52	0.52	0.52	0.36	0.36	0.62	2.352	2.355
	46	A	16— 5	1.92	3.06	3.45				2.193	
	49	A	16— 4	1.54	3.46	3.46				2.185	
VIII.	49	B	16—13	1.75	3.16	3.86	1.74	3.23	3.59	2.205	2.194
	58	A	18— 3	0.34	0.69	1.08				2.291	
	59	A	19— 5	0.97	1.29	1.62				2.341	
IX.	60	A	19— 1	0.66	0.98	1.31	0.65	0.99	1.32	2.383	2.338
	67	A	23— 7	0.80	1.60	2.40				2.273	
	68	A	23— 2	0.81	1.35	1.62				2.270	
X.	69	A	19— 3	0.98	1.68	1.95	0.86	1.53	1.99	2.162	2.235
	72	A	14—12	5.31	5.31	6.64				2.165	
XI.	73	A	15— 1	4.98	4.98	5.81	5.14	5.14	6.22	2.171	2.168
	81	A	9— 7	18.54		20.53				1.606	
XII.	81	B	17— 7	12.19		13.62	15.32		17.08	1.777	1.691

It was the intention of the writers to make freezing tests on the various specimens as is done on building stone to see if the alternate freezing and thawing would lessen the crushing strength or decrease the weight by disintegration. The specimens used for absorption, after having been immersed a week in water were put in a freezing box and packed with ice and salt where the average temperature was 12° *F.* After freezing 18 hours they were taken out, put in water again and allowed to thaw 6 hours. This operation was repeated twelve times. After they had thawed the twelfth time they were again weighed and none showed that they had absorbed any more water than at the end of one week with the exception of No. XI which had absorbed about

one per cent more. They were then placed in the drying room where they remained about two weeks, being then weighed to ascertain the loss due to disintegration. But if there was a loss it was so small that it could not be measured by our scales. These specimens were then crushed and no diminution of crushing strength was noticable except in No. XI, both specimens of which broke under those that had not been frozen, though this may have been due to other causes and not to the freezing. Had time permitted, the number of alternations of thawing and freezing would have been increased till the bricks had shown a loss either of weight or of strength. We can find no record of this test having ever been tried on paving brick before and it would make an interesting investigation of itself if carried to its fullest extent. No. XII was omitted from this test but had it been included it would probably have shown more marked signs of disintegration. The only result obtained by this test is a demonstration of the fact that a good paving brick is not easily affected by frost,—probably no more so than most building stones.

Four cubes of each variety were used in the abrasion test, being weighed after having been perfectly dried. The rattler or tumbler in which the tests were made consisted of a wrought iron cylinder whose length is 12 inches and diameter is 6 inches, inside measurements. This cylinder revolves about a horizontal shaft perpendicular to its axis at the rate of 47½ revolutions a minute. In this cylinder the test pieces were placed five at a time together with five one-inch tool-steel cubes, tempered to their utmost hardness and squared off on an emery wheel. Four runs were made for each set of five and after each run the weight was taken again. The first two runs were each 15 minutes in length and the last two were 30 minutes. The tumbling was not of so severe a nature as to break any of the specimens but the loss was due entirely to abrasion. The percentages of loss were computed for 15, 30, 60 and 90 minutes of tumbling. There is also a percentage for the last hour alone. This is given because it is thought to be the most important of the test as a very hard brittle brick will lose more during the first quarter or half an hour than a tough, soft one, but after the sharp edges have been chipped off, it will stand very much more wear. Since all the wear comes on the flat exposed surface when in service, the last hour in the tumbler more fairly represents the conditions which will obtain in the pavement.

ABRASION.

No.	Brick.	Letter.	Wt. before Tumbling.	Percentage of Loss after. 15 m.	30 m.	1 h'r.	1½ h'r.	Last hour.	Averages. 15 m.	30 m.	1 h'r.	1½ h'r.	Last hour.
	1	B	18—12	1.33	2.33	3.83	4.50	2.17					
	3	A	19—11	0.95	1.90	2.86	3.81	1.91					
I.	3	B	11— 9	1.08	1.62	2.43	2.97	1.35					
	3	C	15— 1	1.66	2.28	3.32	3.73	1.45	1.255	2.032	3.110	3.753	1.717
	9	B	18— 1	2.08	2.77	4.67	6.40	3.63					
II.	10	B	16— 2	1.55	2.71	3.88	5.04	2.33					
	12	C	17— 1	1.47	2.20	3.30	4.40	2.20	1.700	2.560	3.950	5.280	2.720
	19	C	12— 3.5	1.28	2.05	3.33	4.35	2.30					
	19	D	16—14.5	1.11	1.48	2.40	3.33	1.85					
III.	19	E	15—13	1.19	1.58	2.77	3.56	1.98					
	19	F	19— 0	0.82	0.99	2.14	2.96	1.97	1.100	1.525	2.660	3.550	2.025
	23	B	14— 8	1.29	2.16	4.09	6.03	3.38					
IV.	24	B	14—10.5	1.71	2.56	4.48	6.18	3.92					
	24	C	19— 5	1.29	2.10	3.56	5.02	2.91					
	25	C	18— 6	1.70	2.38	3.74	5.27	2.86	1.498	2.300	3.970	5.625	3.327
	28	C	13—15	1.57	2.69	4.26	6.05	3.36					
	29	B	14— 0	1.56	2.23	3.57	4.69	2.46					
V.	29	C	18—15	1.16	1.98	2.81	3.63	1.65					
	30	B	14—10	1.71	2.99	4.49	5.77	2.78					
	31	C	15— 4	1.64	2.87	4.30	5.74	2.87	1.528	2.552	3.886	5.176	2.624
	34	D	10—10	1.18	1.76	3.53	5.29	3.53					
	35	B	14— 9.5	1.50	2.57	3.85	5.14	2.57					
VI.	36	B	17— 3	1.11	2.00	3.64	4.73	2.73					
	37	B	9—13.5	1.59	2.54	4.76	6.35	3.81	1.345	2.218	3.942	5.378	3.160
	40	C	8— 1	1.94	2.33	3.88	4.26	1.94					
	41	A	12— 1.5	1.03	1.29	1.81	3.62	2.33					
VII.	42	D	11—13	1.06	1.59	2.12	3.17	1.59					
	42	E	11— 3.5	1.11	1.67	2.51	3.06	1.39	1.285	1.720	2.580	3.528	1.812
	46	D	25—11	1.22	2.07	3.89	5.60	3.53					
	47	B	14—11.5	1.06	1.93	3.18	4.46	2.55					
VIII.	47	C	15—15	1.57	3.12	5.49	7.65	4.51					
	48	A	16— 4	1.54	3.27	4.42	6.15	2.88	1.348	2.598	4.245	5.965	3.368
	60	B	25—15	1.08	2.05	2.53	3.13	1.08					
	61	A	10—10.5	0.59	1.76	2.64	3.52	1.76					
IX.	61	B	17— 8	1.25	2.50	3.39	5.00	2.14					
	61	C	23— 9.5	1.32	1.99	3.05	4.24	2.25	1.060	2.075	2.902	3.972	1.808
	66	B	32— 0	1.37	2.34	3.71	4.98	2.64					
	67	C	18— 1	1.38	2.42	3.81	5.19	2.77					
X.	68	B	20— 6	1.69	2.45	3.53	4.60	2.15					
	68	C	36—12	0.68	1.36	2.21	3.06	1.70	1.280	2.142	3.315	4.457	2.315
XI.	72	C	13—14	1.35	2.25	3.83	5.18	2.93					
	73	C	16—13	1.12	2.24	3.35	4.28	2.42	1.235	2.245	3.590	4.730	2.675
XII.	82	A	17— 9	3.74	6.94	11.74	16.90	9.96					
	83	B	16—11	2.24	4.85	8.40	11.75	6.92	2.990	5.895	10.070	14.325	8.440

The test of transverse strength was made on whole bricks as is always done. The brick is placed in the same position as it is in the pavement, upon two parallel knife edges, six inches apart. The pressure was applied above at the middle through a third knife edge parallel to the other two. This test was made on a 50,000 pound Olsen testing machine. The "modulus of rupture" found in the table is derived from the formula of Mechanics,

$$M = \frac{R\,I}{e}$$

in which R = Modulus of Rupture, I = moment of inertia, e = one half the depth and M = the moment. Substituting $\frac{1}{12}\,bd^3$ for I, $\frac{1}{2}d$ for e and $\frac{1}{4}\,Wl$ for M, the above equation reduces to

$$R = \frac{3\,Wl}{2\,bd^2}$$

in which l, b and d equal length, breadth and depth respectively. Knowing that l is 6 inches and that bd, the cross section, equals a, we have for this special case,

$$R = \frac{9\,W}{a\,d}$$

TRANSVERSE STRENGTH.

No.	Brick.	Length.	Depth.	Breadth.	Area.	Breaking Wt. in lbs.	Modulus or Rupture.	Average.
I.	5	8.5	4.0	2.5	10.0	14330	3224	
	6	8.5	4.0	2.5	10.0	9520	2142	
	7	8.5	4.0	2.5	10.0	12000	2700	
	8	8.5	4.1	2.5	10.25	16250	3480	2886
II.	13	8.5	4.1	2.5	10.25	6180	1323	
	14	8.5	4.1	2.5	10.25	6180	1323	1323
III.	20	8.3	4.2	2.5	10.50	10110	2064	
	21	8.4	4.2	2.4	10.08	7900	1679	1872
IV.	26	8.5	4.3	2.6	11.18	9270	1645	
	27	8.5	4.3	2.6	11.18	14270	2532	2088
V.	32	8.5	4.2	2.4	10.08	9600	2041	
	33	8.5	4.2	2.5	10.50	11900	2428	2234
VI.	38	8.4	4.1	2.4	9.84	10270	2290	
	39	8.5	4.2	2.4	10.08	9170	1949	2120
VII.	44	7.5	3.6	2.0	7.20	8770	3045	
	45	7.4	3.6	2.0	7.20	7660	2660	2852
VIII.	50	8.5	4.2	2.4	10.08	8870	1885	
	51	8.5	4.2	2.4	10.08	4870	1035	
	52	8.5	4.1	2.5	10.25	10840	2322	
	53	8.5	4.2	2.4	10.08	8550	1817	
	54	8.5	4.2	2.4	10.08	9510	2075	
	55	8.5	4.2	2.3	9.66	10170	2256	
	56	8.4	4.2	2.4	10.08	8370	1779	
	57	8.5	4.2	2.4	10.08	9650	2052	1903
IX.	62	8.4	4.2	2.5	10.50	12580	2563	
	63	8.2	4.0	2.5	10.00	9500	2138	
	64	8.2	4.1	2.4	9.84	9400	2097	
	65	8.3	4.1	2.4	9.84	12670	2826	2408
X.	70	9.0	4.0	2.8	11.20	10230	2055	
	71	9.0	4.0	3.0	12.00	8760	1643	1849
XI.	74	8.2	4.1	2.2	9.02	6550	1594	
	75	8.0	3.9	2.2	8.58	10080	2653	
	76	8.2	4.0	2.3	9.20	8420	2105	2176
XII.	77	7.9	3.8	2.2	8.36	1900	538	
	78	7.8	3.7	2.2	8.14	2080	621	
	79	7·6	3.8	2.0	7.60	1250	389	516

The number of cubes used to determine the crushing strength per square inch of the different varieties, varied from four to six. Three cubes were saved for this purpose alone while those cubes that were used in the absorption, specific gravity and freezing tests were also used for this test. The test was made with the Reihlé Bros. 100,000 pound machine. The test pieces were coated with a thin layer of plaster of Paris and set on a self-adjusting bedplate with pieces of zinc between the test piece and the machine both above and below (the only object of the zinc being to protect the plates of the machine from the plaster of Paris). The stress was applied very slowly and uniformly. The specimens varied more in this test than in any other, there being a variation of 3,000 or 4,000 pounds in two specimens of the same variety.

Crushing Strength.

No.	Brick.	Letter.	Height.	Area.	Crushing in lbs.	Crushing lbs. per sq. in.	Average Crushing per sq. in.
I.	1	A	1.65	3.17	30150	9512	
	2	A	1.93	4.52	90000	19912	
	4	A	1.70	4.56	100000 †	21929	
	4	B	2.42	5.48	76500	13960	
	4	C	2.12	6.14	100000 †	16286	16336
II.	*9	A	3.02	4.15	24800	5976	
	10	A	1.95	4.10	67000	16341	
	11	A	1.98	3.93	58530	14893	
	11	B	2.70	3.81	33070	8680	
	12	A	1.97	4.65	48400	10386	
	12	B	2.48	6.10	59760	9797	11012
III.	17	A	2.40	4.35	40670	9349	
	17	B	1.94	4.36	36800	8442	
	17	C	2.78	4.23	24580	5810	
	18	A	1.95	4.01	45060	11237	
	19	A	2.72	4.32	45160	10454	
	19	B	1.87	4.37	45400	10389	9280
IV.	22	B	2.55	4.84	30050	6209	
	23	A	2.42	3.51	23850	6795	
	24	A	2.50	3.33	21060	6324	
	25	A	2.42	3.40	27900	8206	
	25	B	2.50	4.72	36000	7628	7032
V.	28	A	3.22	4.41	------	-----	
	28	B	2.42	3.32	28900	8704	
	29	A	2.38	3.22	28050	8711	
	30	A	3.14	5.28	36070	6832	
	31	A	2.23	3.22	16860	5233	
	31	B	2.25	3.01	15900	5282	6952
VI.	34	A	2.35	6.18	67200	10873	
	34	B	2.30	5.17	49600	9594	
	34	C	3.20	4.29	36600	8532	
	35	A	1.73	7.05	70580	10012	
	36	A	1.82	3.97	47340	11925	
	37	A	3.18	3.98	31600	7940	9813
VII.	40	A	2.03	3.80	27300	7185	
	40	B	2.00	3.81	68230	17905	
	41	A	2.00	2.81	34250	12188	
	42	A	1.95	4.37	50000	11441	
	42	B	2.00	2.83	25500	9012	
	42	C	1.95	4.16	40600	9760	11249
VIII.	46	A	2.38	5.06	41470	8196	
	46	B	2.30	5.57	44180	7932	
	47	A	3.34	5.64	35950	6374	
	49	A	2.34	5.10	53000	10393	
	49	B	2.33	5.33	46200	8672	
	49	C	3.32	5.50	28200	5128	7783
IX.	58	A	2.39	5.38	52760	9807	
	59	A	2.34	5.76	31600	5486	
	59	B	2.40	6.84	100000 †	14620	
	59	C	2.40	5.61	43300	7718	
	60	A	2.27	5.76	41860	7267	
	60	B	2.40	5.43	37500	6906	8634
X.	66	A	2.30	6.06	53150	8770	
	67	A	2.28	7.48	94020	12573	
	67	B	3.33	7.38	70100	9498	
	68	A	2.36	7.24	100000 †	13818	
	*69	A	2.75	5.24	18500	3530	
	69	B	2.30	7.36	70140	9530	961
XI.	72	A	2.11	5.38	64200	11933	
	72	B	2.40	5.59	85850	15357	
	73	A	2.46	4.68	37050	7916	
	73	B	2.48	4.66	62000	13305	12128
XII.	80	A	3.68	7.76	12000	1546	
	80	B	2.14	5.13	5950	1160	
	81	A	1.67	5.53	9000	1627	
	81	B	2.22	7.20	25200	3500	1958

* Uneven bearing. † Did not reach ultimate strength.

Grouping the averages given in the foregoing tables for convenience of comparison, we have the following

SUMMARY OF RESULTS:

No.	Specific Gravity.	% Absorption after 1 week.	% Loss 1 hour Abrasion.	Modulus of Rupture.	Crushing strength lbs. per square inch.
I.	2.408	0.86	1.717	2886	16,336
II.	2.326	1.28	2.720	1828	11,012
III.	2.287	3.27	2.025	1872	9,280
IV.	2.067	5.14	3.327	2180	7,032
V.	2.167	4.44	2.624	2234	6,952
VI.	2.182	2.56	3.160	2120	9,813
VII.	2.355	0.62	1.812	2850	11,249
VIII.	2.194	3.59	3.368	1903	7,783
IX.	2.333	1.32	1.808	2402	8,634
X.	2.235	1.99	2.315	1849	9,619
XI.	2.163	6.22	2.675	2176	12,128
XII.	1.662	17.08	3.440	516	1,958

It can be shown that the pressure on a brick in the pavement rarely exceeds 1,000 pounds per square inch. Hence the table shows that any of the varieties have much more strength than would ever be required if properly laid upon a suitable foundation. For this reason the test for compressive strength is not of so much value in comparing different bricks as are those of abrasion, absorption and transverse strength. Therefore in selecting the best from those tested it is considered advisable to give very little weight to the tests of crushing strength and specific gravity, but to base comparisons more upon the other three tests. It happens that in this case it makes no difference in the final result which method of comparison we use, No. I being first, No. VII second, No. IX third and No. III fourth.

These results, combined with the two very important considerations of availability and cost, furnish all reasons for accepting or rejecting any competing bricks, where the decision is based upon merit and not influenced by jobbery.

TRANSACTIONS

OF THE

Association of Engineers

OF

CORNELL UNIVERSITY

2

1894

TRANSACTIONS

OF THE

ASSOCIATION OF ENGINEERS

OF

CORNELL UNIVERSITY.

VOLUME II. 1893-94.

CONTAINING

ADDRESSES BY NON-RESIDENT LECTURERS, MISCELLANEOUS PAPERS, CONSTITUTION AND BY-LAWS OF THE ASSOCIATION.

NOTE.—This Association is not responsible for any statements or opinions advanced in any of its publications.

ITHACA, N. Y.

JUNE, 1894.

E. R. ANDREWS, Printer, 1 Aqueduct Street, Rochester, N. Y.

CONSTITUTION

OF THE

ASSOCIATION OF CIVIL ENGINEERS OF CORNELL UNIVERSITY.

PREAMBLE.

We, the undersigned, members of the Senior and Junior classes in the college of Civil Engineering of Cornell University, do hereby form ourselves into an Association for the discussion of engineering topics, and the promotion of general information on engineering subjects, and do hereby agree to abide by and sustain the following Constitution and By-Laws:

ARTICLE I.

NAME.

1. This Association shall be known as the Association of Civil Engineers of Cornell University.

ARTICLE II.

MEMBERSHIP.

1. The Association shall consist of Active and Honorary members.
2. All Alumni of this college and all students recognized as upperclassmen, and registered in the college of Civil Engineering, are eligible to membership in this Association.
3. Any eligible person may become an honorary member by a two-thirds vote of the members present at any regular meeting. Such members shall have privileges of active members except those of voting and holding office, and shall be exempt from all dues.
4. The membership fees of this Association for all active graduate members shall be *$3.00 per annum. All money received from membership fees shall be devoted to defraying cost of publication of non-resident lectures delivered before the Association. All other expenses of this Association shall be met by direct tax upon the undergraduate members.
5. A copy of each lecture delivered before this Association shall be forwarded to each member of the Association.

*This fee is to be changed, by amendment, to a more moderate sum for the coming year.

ARTICLE III.

OFFICERS.

1. The officers of the Association shall consist of a President, Vice-President, Recording Secretary, Corresponding Secretary, and Treasurer.

2. The President shall preside at all meetings of the Association and enforce the Constitution and By-Laws, and shall call special meetings at the request of five active members.

3. The Vice-President shall take the chair at the request of the President, and shall act as President in his absence. The Vice-President shall be chairman of the appointment committee.

4. The Recording Secretary shall keep minutes of proceedings of all meetings of the Association and shall post notices for the same.

5. The Corresponding Secretary shall attend to all the necessary correspondence of the Association. He shall be elected from among the Faculty of the college.

6. The Treasurer shall receive all money and dues, and shall pay all bills of the Association, such bills to meet the approval of the Executive Committee before such payments. He shall make a report when called upon by the Association and also when his term of office expires. He shall be Chairman of the Executive Committee.

7. The officers shall be chosen by ballot at the last regular meeting of the spring term, from the Junior Class, and shall hold office until their successors are elected.

ARTICLE IV.

COMMITTEES.

1. There shall be two Standing Committees, an Executive Committee and a Committee on Appointments. Each committee shall consist of three members and be appointed at the beginning of each term by the President.

2. The Executive Committee shall see that the rooms of the Association are ready for occupancy previous to all meetings, and shall transact such business as may be referred to it by the Association.

3. The Committee on Appointments shall make appointments for all literary exercises for each meeting, and such appointments shall be posted at least two days before reading. The committee shall furnish the Secretary with a list of such appointments.

ARTICLE V.

AMENDMENTS.

This Constitution or By-Laws may be amended by a two-thirds vote of all members present at any regular meeting; such amendment to be before the Association at least one week.

BY-LAWS.

ARTICLE I.

REGULAR MEETINGS.

Regular meetings shall be held on Friday of each week, in the Association rooms, commencing on the first Friday after registration week, and ending on the last Friday but one before examination week of each term.

ARTICLE II.

QUORUM.

One-third of the active under graduate members of the Association shall constitute a quorum. No business can be transacted without a quorum being present.

ARTICLE III.

ORDER OF PROCEEDINGS AT A REGULAR MEETING.

1. Roll Call.
2. Minutes of Preceding meeting.
3. Literary Exercises.
4. Unfinished Business.
   a. Report of Standing Committees.
   b. Report of Special Committees.
   c. Report of officers.
   d. Miscellaneous business.
5. New business.
6. Adjournment.

ARTICLE IV.

EXERCISES.

The exercises shall consist of discussions, memoirs, essays, papers, lectures, and such other exercises as the Association shall from time to time direct.

ARTICLE V.

SUSPENSION OF BY-LAWS.

A By-Law may be suspended for one meeting by a vote of two-thirds of the members present.

H. R. LORDLY,
E. J. FORT,
H. D. ALEXANDER,
Committee.

OFFICERS FOR 1893-1894.

PRESIDENT,

H. W. STRONG, '*94*.

VICE-PRESIDENT,

T. S. CLARK, '*94*.

CORRESPONDING SECRETARY,

PROF. C. L. CRANDALL, '*72*.

SECRETARY,

C. M. AYRES, '*94*.

TREASURER,

J. W. TOWLE, '*94*.

COMMITTEE ON APPOINTMENTS,

S. T. NEELY, '*94* (*Chair.*), I. W. BARBOUR, '*94*,
R. B. GOODMAN, '*94*.

EXECUTIVE COMMITTEE,

J. W. TOWLE, '*94* (*Chair.*), R. B. PARK, '*94*,
S. I. KEHLER, '*94*.

PUBLICATION COMMITTEE,

E. H. HOOKER, '*94* (*Chair*), N. CUMMINGS, '*94*,
C. B. SPENCER, '*94*. A. L. COLSTEN, '*95*.

MEMBERSHIP COMMITTEE,

A. H. PLACE, '*94* (*Chair.*), B. F. LATTING, '*94*, C. W. MARSH, '*94*,
G. G. BROOKS, '*94*, W. H. DUNHAM, '*94*,
W. W. HOY, '*95*.

PRESIDENT'S ANNUAL ADDRESS.

Members of the Association of Civil Engineers, Cornell University.

GENTLEMEN:—Twenty years ago the first engineers association at Cornell was formed, and since that time its life has been varied; sometimes strong with a large membership and an enthusiastic support, and again feeble with but little interest shown by undergraduates or alumni.

The records of the various associations have all been preserved and are of interest as showing the growth from the first formation to the present Association which has it in its power to wield a strong influence for good over the students of Lincoln Hall.

Before presenting some suggestions for the ensuing year it may be well briefly to recapitulate some of the events which have occurred during the past year.

As you all know our first meeting in the fall was saddened by the knowledge of the loss of our class-mate, Harry Merrick Beach, whose death occurred during the summer. His strength of character was well known and his loss was felt most keenly by those with whom he was most intimate.

At the beginning of the winter term the meetings assumed their usual proportions and programs were presented which were usually of merit and always served to rouse a strong interest in the subject under discussion.

For the smoothness with which the literary part of the programs was carried out the committee on appointments deserve much praise and I wish here to thank them for the time and labor they have put on what is at best a thankless task. While the papers read before the association were often elementary in scope yet they all showed careful preparation and a desire to appear before the Society with a production worthy of presentation. And it is only by carefully keeping up this spirit of enthusiasm and pride in the transactions of the Association, that you will be able to accomplish any good or in fact keep the Society alive.

For, as soon as the meetings appear in the light of a duty rather than a privilege; as soon as attendance begins to flag, and the programme becomes a matter of form and not of vital interest—just then will begin the decay which will lead the Association into the same grave

with those which have preceded it. And I urge upon you the necessity of attending the meetings and using your best endeavor to keep the Association alive now that it has started again so fairly.

The non-resident lecturers who have appeared before the Association have been well received and their productions enjoyed by many even outside of our college.

Mr. Clemens Herschel, Mr. T. C. Mendenhall, Mr. J. T. Freeman, and others, are men of such note, and engineers of such prominence that the lecture halls were crowded when they appeared; and I am sure that I voice the sentiment of the entire college in saying that we are very grateful when men of such ability, on whose time there are so many demands, can feel sufficiently interested in the engineering profession to devote their time and the benefit of their experience for our instruction.

At one of the meetings during the winter Prof. Church lectured on the "Flight of Birds,"—a subject of great interest at present, and during the Spring term Prof. Fuertes presented a paper on "Roman Aqueducts" which was unique in the manner of treating the subject, and was much enjoyed by all who were fortunate enough to hear it.

An invitation to send a representative to a dinner given by the undergraduates in science at McGill University was accepted, and the delegate wishes to thank the Association for the honor of that position, and to urge the necessity of in some manner returning the compliment during the ensuing year.

The treatment met with at McGill was most cordial and the governing idea of the college at that time seemed to be to entertain as royally as possible. And now in making the following suggestions be assured there are many more which will occur to you on second thought, which will go far toward making the Association more active and beneficial. From the experience of the past year then I would say: Begin your meetings the first Friday after registration day in the Fall, and keep them up regularly during the term. Get up as much discussion regarding the papers as possible; there are always men who can criticise; find them out and use them to draw out the salient points of the problem in hand. By doing this members will become more accustomed to putting their ideas in logical shape and be better able to express themselves intelligibly. Another scheme for securing interest is to have the Association take up some work as part of the regular business, as for example, cataloguing the pamphlets received in the library of our college. Other lines of work will suggest themselves to you, all of which will aid in keeping the Association strong and well cemented together.

And finally I wish to thank the officers Mr. Towle, Mr. Clark, and Mr. Ayres for their faithful services during the year, and also the committee on publication which has labored so diligently to bring out the Transactions in good form.

With much gratitude to the members for the courtesy and consideration shown me during the year, and a strong wish for and presentiment of success for next year, I bid you good-bye hoping that the Cornell engineers who go out from Alma Mater this year, will prove true to her and to this Association.

H. W. STRONG.

May 18, 1894.

Officers for 1894-95

Elected at the last regular meeting, May 18, 1894.

PRESIDENT,

WARNER W. GILBERT.

VICE-PRESIDENT,

ALBERT H. SEABURY.

TREASURER,

WILLIAM W. HOY.

CORRESPONDING SECRETARY,

PROF. CHAS. L. CRANDALL.

RECORDING SECRETARY,

CHAS. H. KENDALL.

APPOINTMENT COMMITTEE,

ALBERT H. SEABURY, *Chairman.* FRANK C. WOLFE,
HARRY C. DeLANO.

EXECUTIVE COMMITTEE,

WM. W. HOY, *Chairman.* MILO S. McDIARMID,
GLENN D. HOLMES.

PUBLICATION COMMITTEE,

ALBERT L. COLSTEN, *Chairman.* CHAS. H. KENDALL,
LORIN H. IRELAND.

ASSOCIATION OF CIVIL ENGINEERS

— OF —

CORNELL UNIVERSITY.

NOTICE TO MEMBERS.

The College of Civil Engineering of Cornell University, receives many applications each year, for young and for experienced engineers, and often for very important positions; so many in fact, that not only the graduating classes, but older graduates are aided to make changes for more desirable fields of work. Sometimes graduates of other schools even, receive recommendations for lack of candidates from Cornell.

By a little effort, this system may be extended, so as to aid, not only the younger classes, but other members who may be out of work, or may desire a change of location or experience; and also, it may enable our older graduates to be more useful to their schoolmates than at present, by availing themselves of the facilities that such a system would offer them to aid Cornellians whose training they know, whose company they would enjoy, and in whose professional growth they would take more interest than any other class of engineers.

Since engineers are often needed "at once" I receive numerous telegrams to supply engineers that I cannot furnish for lack of knowledge, in a majority of cases, of the whereabouts and wishes of our graduates, though generally, "some one" would be glad, or would be benefited by the offers thus suddenly and frequently made.

It is suggested that all *our* alumni send to the undersigned, 1st, their permanent home address, and 2d, their present address and employment, renewing it when accepting a new engagement. Also, those desiring changes, or new positions, would, without doubt, find it desirable to notify this college, stating experience, position preferred, salary expected, etc., etc.; and, if those knowing of available positions or needing assistance would do likewise, the exact information that could be furnished and the good will and influence of the college of the University, may be of material service to our graduates with advantages readily understood by those giving and requiring employment.

With sincere greetings and best wishes for all, I remain, etc.,

E. A. FUERTES, *Director*.

ITHACA, N. Y., June, 1894.

In Memoriam.

HARRY MERRICK BEACH was born at Cortland, N. Y. in 1873. He prepared for college at the State Normal School at Cortland and distinguished himself at his entrance to Cornell by winning the Sibley Scholarship which he held until the time of his death. While in Chicago in September 1893 he contracted a cold which resulted in typhoid fever and in his death after a short illness. His death was an unusually sad one, coming as it did almost on the threshold of his entrance upon a career which seemed filled with promise of professional success. The following resolutions were adopted by the Association of Civil Engineers:

Whereas, the All-wise Creator has seen fit to remove from our midst our beloved friend and associate, Harry Merrick Beach, and realizing that our Society has lost a member whose good fellowship, integrity of character, diligence in duty and ability as a student had won for him our friendship and admiration.

Be it resolved, that we extend our sincere sympathy to the bereaved family of our deceased friend and classmate.

FRANK W. SHEPARD, born at Richfield, Ohio, June 7th, 1862, received his preparatory education at the public schools of his native place and Oberlin College, and was graduated with the degree of C. E. from Cornell University, June, 1886. He was employed in the office of the Ohio Ry. Co. at Akron, Ohio, until January, 1887, when he accepted the position of assistant engineer with the Sante Fé Railroad in charge of the construction of important terminal facilities at Los Angeles, Cal., and the line to the Pacific. At the completion of this work in September, 1888, he went to Oregon and Washington, and during the winter, and spring of 1889 was employed in the city engineer's office at Tacoma. His health failing, in July he returned to his home in Ohio, was under a physician's care until the winter, when he was again engaged on railway

construction in Ohio. In December, 1890, he was employed with the United States Corps of Engineers under Captain Black, in Florida, as a superintendent of River and Harbor Improvement at St. Augustine until May, 1891; at the mouth of the St. Johns river until June, 1892, and died from the result of injuries received from the falling of broken machinery producing fracture of the skull and internal injuries while engaged in the clearing and improvement of the Oklawaha River, February 10th, 1892.

October 2d, 1891, he was married to Miss Charlotte J. Marsh, of New Brunswick, N. J., who, together with college and professional associates, mourn the early separation from one beloved, and close of a career which untiring zeal and great aptitude rendered unusually full of promise.

The Association is called upon to mourn the loss of still another of our graduates in the death of Mr. D. C. SHELDON of the Class of 1883.

Shortly after graduating, Mr. Sheldon received an appointment as assistant engineer of the C., B. & Q. Ry. Co. in Wisconsin, where this company were engaged in the construction of some extensions of their system. Mr. Sheldon remained in Wisconsin until July, 1886, when he was sent by the same company to assist in making a survey from Denver, west, over the Continental Divide. When this survey was completed he located in Denver, from which time until his death he was connected with the engineering department of some of the railway companies centering at that point. While he was connected with the construction of some difficult mountain work he was attacked by the disease which resulted in his death. He had the reputation of being a good engineer, and was a conscientious man in all his dealings. He was possessed of the qualities which go to make a good American citizen, and those who knew him deeply felt the loss occasioned by his death. He died at Denver, Colo., on the 2d day of October, 1893. When a student at Cornell his home was near Syracuse, N. Y.

FRONTINUS, AND HIS II BOOKS

ON THE

WATER SUPPLY OF THE CITY OF ROME.

A. D. 97.

A LECTURE DELIVERED BEFORE THE ENGINEERING STUDENTS OF CORNELL UNIVERSITY, FEBRUARY 2nd, 1894.

By CLEMENS HERSCHEL, HYDRAULIC ENGINEER,

OF NEW YORK, N. Y.

Quum Cerealis quidem alterius successoris curam famamque obruisset: sustinuit quoque molem Julius Frontinus, vir magnus, quantum licebat, validamque et pugnacem Silurum gentem armis subegit, super virtutem hostium, locorum quoque difficultates, eluctatus. Tacitus, in Vit. Agricola. C. 17.

Now the conduct of Cerealis was sufficient to obscure the fame of his successor, but Julius Frontinus, a great man, sustained the full glory of Cerealis, at every opportunity, and subdued the powerful and warlike people of South Wales, in whom he had to surmount not only a determined courage, but also the difficulties of their country."

"Mihi vero etiam illud gratulatione dignum videtur, quod successi Julio Frontino, principi viro." Pliny, Epist. 4. 8.

"What recommends this dignity (that of augur), to me still more is, that I have the honor to succeed so illustrious a person as Julius Frontinus."

"Impensa monumenti supervacua est; memoria nostri durabit, si vita meruimus." Pliny, Epist. 9. 19.

"The expense of a monument is superfluous; the remembrance of me will remain, if my actions deserve it."

Frontinus' own words, quoted by Pliny.

ITHACA, N. Y.,

1894.

Frontinus, and his II. Books on the Water-Supply of the City of Rome.

Original Sources of Knowledge.

About equally distant from Rome and from Naples, and not far to the eastward of a line drawn from one to the other, is, or was until 1866, the celebrated Benedictine Convent of Monte Casino.[1] The race of monks who inhabited this monastery, from the year 529, when it was built by St. Benedict himself, down to 1866, when the corporation was dissolved by law, has almost disappeared, although a few remain to guard its literary and other treasures. Among these is yet, so far as I know, the only original manuscript of a work entitled: I give it in translation: "The II. Books of (Sextus) Julius Frontinus on the water-supply of the city of Rome,[2] being a manuscript of the 13th, or of the first part of the 14th century. Monte Casino had been destroyed by the Lombards, Saracenes, and Normans in 589, 884, and 1030, and by an earthquake in 1349, but this and other treasures survived the wrack and ruin of eight or nine centuries, to be discovered there, about A. D. 1400, by that indefatigable seeker and disseminator of recorded knowledge, Gian Francesco Poggio Bracciolini, ordinarily called Poggio. We infer, that notwithstanding the great age of the Monte Casino codex, it can present only approximately the precise form in which Frontinus wrote his commentary. Such manuscripts could not have lasted many centuries in troublous times, and a manuscript work composed in 97 and found in 1400, is already the result of probably half a dozen reproductions with the pen. From about 1400 to the year 1459, when he died, Poggio found, copied and distributed, to various libraries, many copies of ancient manuscripts, which I will not stop to describe. Of Frontinus, he made eight copies, which have formed, together with the original codex, the basis of many printed editions.

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[1] See Mackey's Life of Bishop Forbes, 1888.

[2] Mabillion, Museum Italicum. Part I., p. 121.

There is a translation into German, and another into French, but none into English. I may, however, say, that should my plans be carried out, there will be one before many years.

In the sleepy German town of Wolfenbüttel, is a famous library, containing 300,000 volumes and 7,000 MSS. There does not seem to be much of anything else of interest at Wolfenbüttel, and only 13,500 inhabitants, but they appear to prize that library, and have put it for safe keeping into a new building so recently as 1887. Among those 7,000 MSS. is one of the oldest Latin ones in existence, called the Codex Arcerianus.[1] It is supposed to have been written, possibly in the 6th century, certainly not later than the 7th. It appears to have been a book used by the Roman State employees, to aid them in the discharge of their duties, and contains, besides treatises on Roman law, much about land surveying, as taught by several Roman *agrimensores*, or *gromatici*, or land surveyors; among the lot, some pages by Sextus Julius Frontinus.

We thus find Frontinus one of that band of modest land surveyors, of whom it has been said in modern times, that in their work of conservation of the mathematical arts, across the dark and into the middle ages, they were the humble co-laborers of the Arabian scholars of the time, and that to them is due the credit of having saved the Geometry of the ancients for the benefit of the earliest years of the renewal of learning, and thus for succeeding ages.[2] The writings of all these men bear distinct traces of the influence upon their authors of the teachings of Hero of Alexandria, a contemporary of Vitruvius, about 100 B. C.; and on the other hand, Book 2 of the Geometry of Anicius Manlius Severinus Boethius, born 480–482, and beheaded 524, (now known in the Saint's Calendar as St. Boetius), is held to have been founded on, and to be a partial reproduction of, the treatise on surveying of Frontinus.[3]

In the St. Peter's Convent of Salzburg, in the Austrian Alps, is a manuscript treatise on Geometry, by Gerbert, who afterwards became Pope Sylvester II., and who wrote in the closing years of the 10th century. Some student of this codex, writing in the 12th century, has made a marginal note, which shows that he knew, and had read, the

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[1] See: Die Römischen Agrimensoren. Moritz Cantor, 1875, p. 95. Or, Vorlesungen ü. Geschichte d. Mathematik, by the same author, p. 467.

[2] Agrimensoren, p. 185.

[3] Cantor, Gesch. d. Math.

full treatise of Sextus Julius Frontinus on land surveying, described and given in part, in the Wolfenbüttel Codex Arcerianus, just spoken of.[1]

Another old MSS. kept at Chartres, in France, by an unknown author, is believed, by so careful a student as Chasles of the French Academy, to be a portion of this lost treatise on land surveying of Frontinus.[2]

Finally, we have had come down to us a treatise on Strategy, by Sextus Julius Frontinus, much prized by military authors, and of which there is an English and a French translation.[3] Frontinus is also mentioned and praised by many ancient writers, such as Tacitus, Aelian, Vegetius, Martial, Pliny and Suetonius.[4]

These are, so far as I have been able to learn, the existing original sources of our knowledge concerning a man, but for whom we should now know very little about the construction and operation of hydraulic works, 1800 years ago, or of the state of knowledge of applied hydraulics, at that time. From them we can construct a sketch of the life of this land surveyor, soldier, and water works commissioner of ancient times, which may properly precede a discussion of his description of the water-works of ancient Rome.

SEXTUS JULIUS FRONTINUS.

Sextus Julius Frontinus is supposed to have been born about the year 40 of the Christian era, and to have died in 103.[5] He was a public officer during the reigns of Vespasian, Titus, Domitian, Nerva and Trajan. Julius Cæsar was already about as dead to him as he is to us. On the other hand, he had not the remotest idea that such a thing as the fall of the Roman Empire could or would ever take place. This will give us a sort of measure by which to determine his position on the shores of the stream of time. It was under Vespasian, 69–79, that a survey of all lands was ordered, and it is supposed that Frontinus was engaged on this work, and then wrote his treatise on surveying, of which mention has been made. A. D. 70,[6] he was Praetor Urbanus.

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[1] Agrimensoren, p. 94, 202.

[2] Cantor, Gesch. d. Math. p. 500.

[3] See: "Stratagems of War" by Sextus Julius Frontinus, Robert Scott, London, 1816; and "Stratagematicon, et Recherches sur la vie de Frontin," Paris 1772.

[4] Tacit. in Vit. Agricola. C. 17; 1 Hist. Lib. 4, C. 32; Martial. Lib. 10. Ep. 48 and 58. Pliny Epist. Lib. 19, and elsewhere.

[5] Pliny Epist. 19.

[6] Tacit. 1 Hist. iv. 32. Suet. Vit. Domitian, C. 1.

He was governor of Britain, 75–78, commanded an army-corps, and subdued the Silurii, inhabiting what is now Wales; a highway in Monmouthshire, built by him, still bears his name, the Julian road; and certain passages in his work on strategy cause it to be believed that he fought in Germany also, with Domitian, the son of Vespasian, 81–96.

The emperor Nerva, reigned only from Sept. 19th, 96, to Jan. 28th, 98,[1] and as new brooms in politics probably swept cleaner then than they now are allowed to do, and to the victors belonged the spoils, much more than they do now, it is not unreasonable to suppose, that it was not long after Sept. 19th, 96, that Frontinus was made Curator Aquarum, or sole imperial water commissioner of the water-works of Rome; in which position we find him in 97. This was a highly responsible office as can readily be imagined. Instituted before A. D. 14, say A. D. 10, Frontinus had had 16 predecessors in office; so that their average term of service had been only five years four and one-half months.[2]

Frontinus was also Consul Suffectus in 97 and Consul in 100, under Trajan.

Passing now to his treatise on the aqueducts of Rome, he says himself that he wrote it almost immediately on taking the office of water commissioner. Let us let him speak now upon this and other points, that we may gather from his own lips what manner of man this was. This is the way the commentary begins:

"1. Inasmuch as every office, conferred by the emperor, demands especial attention; and inasmuch as I am moved not only to devote diligence but even love of work towards these to me confided duties; be it on account of in-born zeal, or by reason of faithfulness in office; and as Nerva Augustus, an emperor of whom it is difficult to say, whether he devotes more love or more diligence to the common weal, has now imposed upon me the duties of *curator aquarum*, duties contributing partly to the uses, partly to the health, even to the safety of the city, and from olden time exercised by the most distinguished citizens; I therefore consider it to be the first and most important thing to be done, as has always been one of my fundamental principles in other affairs, to learn thoroughly what it is that I have undertaken.

2. Because to my mind there is no better foundation for any business, nor can under any other conditions be decided what is to be done, and

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[1] Duruy's History of Rome.

[2] Frontinus, 99 and 102.

what omitted; nor is there for a fair minded man so debasing a course, as to fulfill the duties of an entrusted office according to the directions of assistants; which course, however, must be run, whenever there is ignorance of the duties of an office, by him, who, frequently, thus craves the practical experience of his assistants; whose services, though necessary in the conduct of the subject matter should, nevertheless, be only a sort of hand and tool, of the principal in charge. It is for this reason that I have set down in this commentary all that I could gather as bearing on the subject matter, after having arranged and codified it in accordance with my habit, as already exemplified in other offices, so that I might consult it as a guide to the duties of this office. But my other commentaries were inspired by my own hard earned practice and experience, and were intended for the benefit of my successors, and this commentary may also be of use to a successor, but as it has been written at the beginning of my administration, it will serve more especially for my own instruction and as a guide."

17. "It has seemed to me, not improper, to examine consecutively, the lengths of channel of each aqueduct, in its several parts and in detail. This because the maintenance of the works, is the most important part of the duties of this office, wherefore it is necessary that the incumbent know, which of them are in need of extended repairs. But our zeal was not satisfied by a mere personal examination in detail; we also caused to be made plans of the aqueducts, from which may be seen, where there are valleys, and where rivers have been crossed, and their widths, also where the conduits laid on the hill-sides need an extended and lasting care for their protection and maintenance. In this way, we reap the advantage of having, as it were, the works referred to directly before us, and of being able to discuss them as though we stood beside them."

23. "Having now given the builders and the age of each aqueduct, also their sources, lengths of channel, and order of heights, it seems to me not improper to go more into detail, and to determine how large is the quantity of water which is applied to public and to private uses, as well as to their respective pleasures, and through how many cisterns, and in what wards these are located; how much water is distributed within the city walls; how much without; how much for water-basins; how much for fountains; how much for public structures; how much on account of the state, how much to private consumers."

As we shall see, he goes on fighting waste, fraud, and unlawful taking of water, in every form, thence on.

64. "It appeared that 12,755 *quinaria*, in all, had been granted; 14,018 had been set in place; that is, 1263 more were discharging water than had been granted. Alarmed at this state of affairs, I felt myself urgently called upon, inasmuch as an important part of my duties, as I believed, consisted in gaining reliable data concerning the water and its volumes, to examine how it was that more could be distributed, than had, so to speak, been inherited. I commenced therefore to guage the aqueducts &c."

A law had been passed, requiring lictors to accompany the *curator aquarum*, on his journeys outside the city walls; says Frontinus: 101, "During my examination of the aqueducts, my spirit of self reliance, and the standing given me by the emperor's commission, shall take the place of the lictors."

103. "I will now set down what the *curator aquarum* must observe, being the laws and Senate enactments which serve for his guidance. As concerns the draft of water by private consumers, there is to be noted: 'that no one may draw water without a grant from Caesar, that is, that no one may draw water from the public supply without a written license, and no one more than has been granted.' By this means, we propose to make it possible, that the quantity of water to be recovered, may be distributed to new fountains and for new grants. But in both cases must a great zeal in action be opposed to manifold forms of fraud. The channels of the aqueducts, without the city, must be frequently examined, one after the other, to locate the granted quantities, the same must be done in case of the cisterns and fountains, that the water may flow without interruption, day and night, which the curator has been directed to see to, by vote of the Senate, which reads as follows: &c."

119. "Many and extended works of repair or of improvement, constantly arise, which must be attended to before they call for large appropriations. As a rule, however, they are only to be taken hold of after due consideration; because, those who urge the construction or extension of works can not always be trusted. The Curator, therefore, must not only cause himself to be instructed by the skill of experts, but must be armed also with self-acquired practical experience. He must consult, not only the builders in the employ of the office, but must seek aid from the trust-worthiness and thorough knowledge of outsiders, in order to judge what must be taken in hand forthwith, and what postponed; again, what is to be carried on by public contractors, and what done by the day."

123. "No class of work demands greater care than that which is

to hold water. Conscientious dealing throughout all the several parts of such works, according to the rules of the several trades, something that is known to all, but followed by few, is therefore a prime requisite."

Referring to a law passed 39 B. C., which made the damaging of aqueducts a misdemeanor, punishable by a fine of 100,000 sestertiae, say $4,000 or $14,000, according as the intrinsic or purchasing value of money be considered:

This law was called "Lex Quinctia," and Frontinus says: 130. "I should call the transgressor of so beneficent a law, not unworthy of the threatened punishment. But those who lived in an atmosphere of delusion, and to whom a violation of the law had become second nature, in the course of time, had to be brought back to the right way of thinking by gentle means. We, therefore, endeavored with diligence, that as far as possible the erring ones should remain unknown. Those who sought the Emperor's pardon, after due warning received, may thank us for the pardon granted. But for the future, I would wish that it might not be necessary to invoke the law, although it is preferable to pursue the duties of one's office, even at the expense of encountering violent opposition."

The Aqueducts.

I spare you from the detailed statistics of the water-works of Rome as given by Frontinus, and to be found in all the encyclopædias. I think it has been belittling to the fame of Frontinus, and our misfortune, that up to the present time, most authors and encyclopædia writers have confined themselves to this branch of the subject. The reason of this may be, that the average antiquarian is not fitted to appreciate or even to discern and repeat the engineering features of Frontinus' writings; any more than he is to describe in detail to engineers, the construction and operation of the water-works of his own time. I assume that I am speaking to an audience conversant with, at least, the general principles of hydraulics, and of water-works construction, as now understood, and proceed accordingly.

Many writers have expressed the opinion that the ancient Romans were engineers, rather than architects; that they taught the useful, rather than the beautiful. Certain it is, that they felt comparatively little predilection for "pursuing science for science sake" as the phrase goes, while they pursued to the utmost of their abilities, that "art of directing the great sources of power in nature, for the use and convenience of man," which constitutes the profession of a civil engineer.

Frontinus gives us a good example of this. After enumerating the nine independent aqueducts, or eleven, if we count two branch aqueducts, which supplied Rome in his day, and giving the history, length above ground and under ground, and other details of each, he exclaims: 16. "Will anybody compare these wonderful works, serving so many needs of man, with the idle Pyramids, or with those other useless, though much renowned works of the Greeks!" But he had considerable reason for self-satisfaction. At a time when the unit of value was a coin of about 4 cts., metal value, when a laborer received about 16 cts. per day, a mason 70 cts. and their board and lodging, rated at about 20 cts. additional, a single aqueduct, "Marcia," had cost about nine million dollars. Altogether, there were over 250 miles of masonry conduit, 220 miles under ground, and 30 miles on arches. One of these rows of arches was 109 feet high.

Eventually, there were fourteen aqueducts, bringing to Rome the water of eighteen springs, or rivers, distant from 7½ to 44 miles in straight lines. The lengths of these aqueducts varied from 11 to 59 miles, and aggregated 359 miles, of which 304 miles were below ground and 55 miles on arches.[1]

Let us now get a clear idea why the Romans built these long and high aqueducts. Let us stamp out, if we can, the shallow notion that those men did not know that water would rise as high in a pipe as the source from whence it came. This can be disproved so easily that it becomes a marvel that so false an idea should continue to demand notice. Here, for example, is Vitruvius, a Roman builder, the predecessor of Frontinus by about 100 years, who wrote ten books on architecture, between 16 and 13 B. C. He tells us how to build what are now called inverted siphons. He says it may be done with lead pipe, or with drain pipe. If of lead and 10 feet long, make them weigh 12 lbs. per inch in circumference; that is, make them uniformly a little over $\frac{5}{16}$″ thick. As he makes them 1¼″ to 24″ in diameter, he leads people into making 24″ pipe, that will stand only about 41 ft. head, while the 1¼″ pipe should stand nineteen times that head. These pipes, as we shall see, were all soldered, not seamless drawn, but the strength of the joint was greater than that of the metal. They could, however, be surrounded by masonry, whose weight could be depended on to increase the strength of the siphon. That this was done, both in the case of the lead, and of the drain-pipe siphons, we know from

[1] Lanciani, Rodolfo, Ancient Rome.

the remains of both kinds which have been found: as for example, the lead pipe siphons at Lyons, France, being nine parallel lines of pipe, 12″–18″ in diameter, and 1″ thickness of metal,[1] under 200 ft. head; a drain-pipe siphon at Alatri, in Italy, built by Betiliemus Varus, 150 B. C., and built to withstand some 300 ft. head.[2] Vitruvius tells us, how in laying such pipe, the mortar at the joints should be made of lime and oil, what is now called "pointing up" mortar, and used to finish the joints on the outside of buildings; that at the angles should be placed a bored-out block of stone; that such siphons must be filled very slowly; and that it is a good plan to put ashes inside, to begin with, so as to stop fine leaks; a little trick of the trade called "puddling with ashes," which, in the case of canal gates and the like, is the common New England practice, about 1900 years after Vitruvius.

Vitruvius learned much that he knew of the Greeks, and in speaking of inverted siphons, he states what the Greeks called them. Remains of Greek drain-pipe siphon aqueducts have also been found in Asia Minor,[3] and are shown in modern books of travel.

In the Natural History of Pliny the Elder, written before A. D. 79, (31–6), it is said in plain language, speaking of water: that "it climbs to the heighth of its own origin": "subit altitudenem exortus sui." So also says Frontinus: 18. "The several aqueducts reach the city at different elevations, whence it comes that some flow on higher ground, while others cannot elevate themselves to the more lofty situations; for the hills, at present, are higher than they once were, on account of the accumulation of rubbish produced by the frequent fires. There are five aqueducts whose waters raise themselves up to all parts of the city, though some are forced up by a greater, others by a lesser head."

It is also inconceivable that any people who had once used flexible lead pipe, should long have remained in ignorance of an inverted siphon.

Nor is it difficult, on the other hand, to find the reason why, knowing all about how to build inverted siphons, the Romans made so seldom use of the principle, in the construction of their main conduits. They did simply what every engineer does at the present day, when working in different parts of the world. At each point of operation, a good engineer will use to the best advantage possible, the material and facilities for his work, found at hand, for the time being, and this is what

[1] Leger, Alfred, Les Travaux Publics aux Temps Romains, 1875.

[2] Lanciani, Ancient Rome.

[3] Belgrand, Les aqueducs Romains, 1875.

the Romans did. Not having cast-iron pipes, they builded as could best be done, without them; and were we deprived of cast-iron, wrought iron and steel pipes, we would to-day, be obliged to build water-works pretty much as they built them.

As regards the source of supply, we might not be so particular as they were. As far as possible, they avoided the taking of river water, and instead, sought far and wide for underground springs. To this day, in countries, which were formerly Roman provinces, the best water to be had, is that which yet flows through some old Roman aqueduct. At Belgrade, in Servia, a friend writes me, the old Roman public fountain is now several feet below the level of the surrounding public square. Owing, no doubt to the calcareous deposits in the conduit, for the water is very hard, only a small stream of water issues forth, to get which, a long line of women, with pails and tubs, patiently await their turn, throughout the day.

When a river was taken, it was first allowed to pass through one or more artificial lakes, so as to deposit the coarser suspended matter, or to maintain its level at the head of the aqueduct; as in the case of the two conduits bringing the waters of the river Anio to Rome.

The masonry conduits had vertical side-walls, and either a ridge roof, made of two stone slabs, or a flat stone cover, or a semicircular arch as a cover. When on arches, they were lined inside, to make them water-tight, with a species of concrete, made of crockery fragments (of which great quantities were in constant demand), and of the Roman hydraulic cement of the time, made of lime and powdered volcanic pumice stone, now called "pozzuolani."[1]

At irregular intervals there is a marked bend in these aqueducts, the purpose of which has long troubled the antiquarians. I suggest the same explanation that explains the double reversed curves, which an antiquarian could find, though not much talked about, in the centre of some of our modern tunnels; also not unknown on railroads, here and there; namely, that the work was commenced and carried on simultaneously at many points, and these bends are where the different parts of the work finally met after undergoing the vicissitudes of hasty construction; such as the difficulties of securing the rights of way originally desired; changes made in the plans during construction; or later, during thorough repairs; and, finally, errors made in alignment. Every 240 ft. is a "lumina," or air hole, to let air in and out, in emptying and filling the conduit; and, it is supposed, also acting as overflows.

[1] Middleton, J. H., Ancient Rome in 1888. Black, Edinb.

There is no regularity in the spans of the arches, nor even in the cross-sections of the piers, but this has been remarked in other engineering works of a much later period, down even to the 16th century.[1]

I intend to say more of the construction of these aqueducts, in dealing with the matter of their maintenance.

The slope of the aqueducts was exceedingly irregular, as might be expected, although there were employees who did nothing but take levels; the forerunners of the wielders of the rod and level of to-day. The methods and instruments they used, are described in works that have come down to us, but I will not stop to repeat the description. Given a knowledge of a plumb line, and the fact that a level line is perpendicular to it, also that water stands level in a groove and in a bent pipe, and given the use of sights to run in lines, and any intelligent mechanic could construct for himself, and use, an ancient Roman leveling instrument. Vitruvius had advised slopes of 1:200, but Frontinus makes fun of this,[2] calls it the way of the ancients, much as we would look down upon the ways of the colonial period in this country; and says that in his day, the art of leveling was better understood, and slopes of aqueducts were made flatter. Marcia, built 145 B. C., is reported to have had a slope of 1:435,[3] at least, in some parts of it; other portions were on a slope of 1:680; and these two slopes are also reported as found in Appia, Anio Vetus, Julia, Virgo, and Anio Novus. Claudia, finished A. D. 50, had a slope of 1:333. Other Roman aqueducts have slopes ranging from 1:3000 to 1:600,[4] some having a very uniform slope, and some showing plainly the difficulties had in sighting in small slopes with poor leveling instruments.

"The ancients," says Frontinus, 18, "laid out their works with greater declivity downwards, be it because the art of running levels, had not yet been developed, so as to attain accuracy; or because they purposely sought to place the aqueducts underground, so that they could not, as frequent wars were still had with the Italians, be as easily cut off by the enemy. But now, should one of the old conduits become useless by age, it is no longer led around the head of the valley underground, but to save distance, is carried across on a foundation-course, or on arches." Little dreaming, that for hundreds of years after his

[1] Belgrand.

[2] Frontinus, 18.

[3] No reliable or expertly determined slopes of the aqueducts of Rome appear to be given in literature on the subject.

[4] Leger; and Belgrand.

time, what he called the "Empress of the world" and the "Eternal City," inclusive of the aqueducts of which he was so fond and proud, was to become the prey of the barbarians.

The nine aqueducts of Frontinus' time, given in order of their age, were called Appia, Anio Vetus or Old Anio, Marcia, Tepula, Julia, Virgo, Alsietina, Claudia, and Anio Novus, or New Anio.

Marcia and Claudia, run practically between the same termini; Marcia was built 145 B. C., Claudia 195 years later. In 1870, the sources of Marcia, flowing in the old subterranean conduit, were sought for and discovered, and its waters were then again brought to Rome; for the greater distance in iron pipes. Now the lengths of these three aqueducts thus built, 145 B. C., A. D. 50 and A. D. 1870, were respectively, 305,500, 230,000, and 173,000, ft.;[1] which figures illustrate very well the resultant distance between two given termini, by the use respectively, of masonry aqueducts ignorantly, and when comparatively well laid out, and of cast-iron pipes, for crossing valleys.

Tunnels.

But, let it not be supposed that the Romans did not know how to build tunnels to shorten distance. Plenty of such tunnels remain to this day, both of the kind that were built in deep trenches and buried up again, (I have but to mention the Cloaca Maxima); and of the kind that were built by tunneling. Duruy's History of Rome, speaks of a tunnel, 16,666 ft. (over three miles) long, in line of the aqueduct of Antibes in France; and of another, to drain Lake Fucinus, in Italy, about A. D. 50, on which 30,000 men worked eleven years and built a tunnel 18,000 ft. long, through rock and clay, 86–96 square ft. in section. There were thirty-two shafts, 65 to 425 ft. deep, and also inclined shafts or slopes, used in hauling out the excavated material. But the best idea of Roman tunnel work can be got from Lancianis's Ancient Rome, in some letters and reports that have been preserved. The correspondence commences thus, A. D. 152:[2]

Varius Clemens, governor of Mauritania, to Valerius Etruscus, governor of Numidia. "Varius Clemens greets Valerius Etruscus, and begs him in his own name and in the name of the town-ship of Saldae, (Algeria) to dispatch at once the hydraulic engineer of the 3rd. legion, Nonius Datus, with orders that he finish the work, which he seems to have forgotten."

[1] Belgrand.
[2] Lanciani, Ancient Rome, p. 61.

Report of Nonius Datus to the magistrates of Saldae:

"After leaving my quarters, I met with brigands on my way, who robbed me of even my clothes, and wounded me severely. I succeeded, after the encounter, in reaching Saldae, where I was met by the governor, who, after allowing me some rest, took me to the tunnel. There, I found everybody sad and despondent, they had given up all hopes that the two opposite sections of the tunnel would meet, because each section had already been excavated beyond the middle of the mountain, and the junction had not yet been effected. As always happens in these cases, the fault was attributed to the engineer, as though he had not taken all precautions to insure the success of the work. What could I have done better?

"I began by surveying and taking the levels of the mountain; I marked most carefully the axis of the tunnel across the ridge; I drew plans and sections of the whole work; which plans I handed over to Petronius Celer, the governor of Mauritania; and to take extra precaution, I summoned the contractor and his workmen, and began the excavation in their presence, with the help of two gangs of experienced veterans; namely, a detachment of marine infantry and a detachment of Alpine troops. What more could I have done? Well, during the four years I was absent at Lambæse, expecting every day to hear the good tidings of the arrival of the water at Saldæ, the contractor, and his assistant, had committed blunder upon blunder; in each section of the tunnel they had diverged from the straight line, each towards his right, and had I waited a little longer before coming, Saldæ would have possessed two tunnels instead of one." Nonius Datus, having re-surveyed the work, caused the two parallel tunnels to be united by a transverse tunnel; the waters of the river could finally pass the mountain, and their arrival at Saldæ was celebrated with extraordinary rejoicings in the presence of the governor, Varius Clemens, and the engineer."

In Rome is Monte Affiliano, and through it one of the longest tunnels built by the Romans, about 3 miles long; L. Paquedius Festus was the contractor, and it was ordered by Domitian. The cross section was 7′ x 3′. The contractor made a vow to restore a temple on the mountain if the enterprise should succeed. The headings met July 3d A. D. 88, and the beautiful columns and fragments of statuary discovered on Monte Affiliano prove that the vow of L. Paquedius underwent a specific performance of its obligations.[1]

[1] Lanciani, Ancient Rome.

THE WATER OF THE ROMAN AQUEDUCTS.

The waters of the several aqueducts had their distinctive characteristics, some being purer than others. All of them were hard waters, of 18 to 27 degrees of hardness; whereas, the water delivered to American cities has seldom so much as 5 degrees of hardness. The deposit left by these waters on the outside of the aqueducts, at points of leakage, was likened, by one old writer,[1] to a hay-stack, which is indicative of the amount of lime contained in the water. Hadriana, built after Frontinus, was ⅔ filled up inside by lime deposits. This deposit also formed inside of the lead pipe used.[2]

Some of these waters are almost lovingly spoken of in ancient literature. The poet Martial, a contemporary of Frontinus, makes frequent mention of the aqueducts. Epigram 10.58 is addressed to Frontinus, the very Sextus Julius Frontinus, whose essay we are considering. Epigram 6.42, he says:

"If the Lacedæmonian customs please you, you may, after a sufficiency of dry heat, plunge into the waters of Virgo, or of Marcia; which shine so brilliantly, and are so pure, that you would scarcely suspect the presence of water, and would imagine that you saw but the polished Lygdian marble."

About the year 500, Theodoric, emperor of the Eastern Empire, writes from Rome to his prime minister, Cassiodorus: "See how the aqueducts of Rome contribute to her ornamentation! Virgo's stream is so pure that the name, according to common opinion, is derived from the fact that those waters are never sullied; since, while all the others give evidence of the violence of rain-storms by the turbidity of their waters, Virgo alone ever maintains her purity."

But Frontinus' account gives the origin of the name "Virgo" to the tradition, commemorated in his day by a mural painting at the spring house, that a young girl showed the soldiers sent to find a spring of water, where to dig for Virgo.[3]

Strabo (5.3–13), speaks of the Marcian water, "which is drunk in Rome in preference to any other;" and Tacitus (A. 14, 22), tells how Nero once polluted its sources. "Nero entered," he says, "for the purpose of swimming in the fountain head of the Marcian water, which is conveyed to the city. He was considered to have polluted the sacred

[1] Fabretti, Raphael.

[2] Belgrand.

[3] Frontinus, 10.

water and to have profaned the sanctity of the place by washing his person there; and a dangerous fit of illness which followed, left no doubt of the displeasure of the Gods;" or of the chill Nero got by bathing in water of a temperature of about 50 Fahrenheit.

In bathing at a spring, Nero violated the public sentiment of the Romans, which considered springs as something sacred. Says Frontinus, 4: "From the foundation of the city, for 441 years, the Romans were content with the use of the waters, which they drew, either from the Tiber, or from wells, or from springs. Springs have held, down to the present day, the name of holy things, and are objects of veneration; having the repute of healing the sick; as, for example, the springs of the Muses, of Apollo, and of Juturna."

Lanciani tells us that the basins of some springs have been excavated in recent times and have yielded layer after layer of precious objects, thrown into them as votive offerings, and representing, consecutively, the handiwork of many centuries.

Alsietina furnished water of so bad a quality that it was used, as a rule, only to supply the basin of a marine circus, and for the irrigation of gardens.

The Distribution System.

These aqueducts ended at various points and at various elevations within the city, generally in form of a large cistern, called *Castella.* Thence were laid lead pipe to other cisterns, for public and for private consumers, to fountains, water-basins, and elsewhere. Thus, *Castra,* were the military camps, or barracks. *Opera publica,* the public buildings or other structures, exclusive of those belonging to the emperor. These last were termed "*In nomine Cæsaris.*" *Laci* and *salientes* were public basins and jet fountains. *Munera* were the large ornamental fountains. *Beneficia Cæsaris* were the imperial grants to private consumers. Says Frontinus, 115: "This mode of gaining money, practiced by the watermen, is also to be abolished, the one called tapping. Far away, and in all directions, run the pipes under the city pavements. As I have learned, these have furnished water by special pipes to all whom they pass and who have been able to arrange for it; being bored for that purpose here and there, by the so-called tappers; whence it came, that only a small quantity of water reached the places of public supply. The amount of water gained in consequence of our abatement of this evil, I measure by means of the fact that we have gathered a large quantity of lead by the removal of that kind of branch pipes;"

which gives a vivid picture of the common and extensive use made of lead pipe at the time.

These lead pipe have been dug up literally by the ton. So late as 1878, Prince Torlonia melted down a ton of them, dug up on his land alone. But in so doing he deprived modern antiquarians of a great treat.[1] Prof. Rodolfo Lanciani, of Rome, has made a special study of these Roman lead pipes, and by means of the inscriptions, in raised letters, ordinarily found upon them, and evidently produced by engraved rollers used in rolling the lead plates out of which the pipes were made, has made some curious discoveries. He has located by means of them the residences of 80 or 90 distinguished citizens. He also finds that there were female plumbers in ancient Rome as well as female householders; but whether a female plumber in ancient times was any more reliable than the male plumber of modern times, the records so far discovered do not say. As already stated, these lead pipe were made by bending lead plates, of the proper width, and some 10 ft. long, into a pear-shaped cross-section, or something like the Greek letter "omega," then soldering the longitudinal joint. The solder used was pure lead. That used in pipes dug up in Lyons, in Paris, of the 2nd century, as well as that used in Pompeian pipes, contained not a trace of tin.[2] I have been unable to find when the modern method of soldering, with an alloy of tin and lead, and with copper points, first came into use. Belgrand, a noted French engineer, caused a lead pipe to be made in imitation of the Roman method of bending the lead plate into a pear-shaped cross-section, and soldering it by pouring melted lead on the joint. He found on testing it, the plate being about ¼″ thick, and the pipe, when rounded, about 4″ in diameter; that at 45 lbs. pressure the pipe began to assume a circular section; at 112 lbs. it was a circular pipe; and it failed at 250 lbs., without failing at the joint. Four way branches, brass stop-cocks, wipe joints, copper bath-tubs as well as marble ones, in short, all the essentials of the outfit of a modern plumber's shop have been recovered in large quantities and may be seen in the museums of Rome and of Naples. And then to talk of the Romans as not having known that water rises in one leg of an inverted siphon when water is poured into the other.

Hydraulics, A. D. 97.

Let us see what ideas were extant on hydraulics, in Frontinus' time.

[1] Middleton.
[2] Belgrand.

The most troublesome point of ignorance he had to contend with was a total inability to measure the velocity of water, or even to rightly and fully grasp the idea of such velocity, whether as flowing in an open channel or in closed pipes. He accordingly compares streams of water merely by the areas of their cross-sections. A square foot of water is all one to him, whether it be one of the 8 square feet of cross-sectional area of a stream in a conduit, or whether it be composed of the sum of 200 or more cross-sectional areas of lead pipes leading out of cisterns to fountains, or to watering troughs, or to private consumers, and ending and discharging at as many different elevations. To the expert of to-day this seems excessively silly; and yet the same thing is constantly being done even now by those who ought to know better. The average man to-day will talk about "a stream of water that will fill a 6 in. pipe," and there are hundreds of deeds on record conveying "square feet of water" for power purposes, just as though the law of falling bodies and its application to hydraulics had never been discovered; and unmindful of the fact, as one old Italian writer on irrigation has expressed it, that to speak of a stream of water by its area of cross-section is like estimating the volume of a cylinder, merely from the area of its base.[1]

These are some of the ideas Frontinus had on the velocity of flow.

35. "Let us not forget in this connection that every stream of water whenever it comes from a higher point and flows into a cistern through a short length of pipe, not only comes up to its measure, but yields, moreover, a surplus; but whenever it comes from a low point, that is under a less head, and is conducted a tolerably long distance, it will shrink in measure by the resistance of its own conduit; so that on these accounts, either an aid or a check is needed for the discharge."

He also says, speaking of Virgo: 70. "The gauging could not be made at the intake, because Virgo is made up of several tributaries, and enters its channel too gently. Near the city, however, at the 2d mile-stone, on the field which is now owned by Cejonius Commodus, and where Virgo has a greater velocity, I made the gauging, and it amounted to 2504 *quinaria*, being 1752 *quinaria* more than was set down in the records. But proof of the correctness of our gauging is at hand; for Virgo discharges all the *quinaria* we found at the point of gauging; that is 2504." Meaning by this that the sum of the areas of all the pipes leading out of Virgo was equal to the area of the cross-section of the stream at the 2d. mile-stone, as found by him; and flattering himself that such equality of areas was the way to attain equality of volumes

[1]Romagnosi, p. 176.

of discharge. He had already said, 65, in speaking of the Appia aqueduct: "at the twin channels, which is below the Old Spes, where it is joined by a branch of the Augusta, I found a depth of the water of 5 feet; a width of 1 foot $\frac{3}{4}$, making $8\frac{3}{4}$ square feet of area; 22 *centenarii*, and 1 *quadragenaria;*" that is:—22 pipes each 100 square digits in area (nominally at least), and 1 pipe of 40 square digits nominal area; "which makes 1825 *quinaria,*" says Frontinus. In point of fact, 5 feet$\times$1.75=8.75 square feet, as he says. This equals 1260 square inches. And as one *quinarium* equals the area of a circle $1\frac{1}{4}$ digits in diameter, it is equal to 0.69026 square inches; and 1260 square inches equals 1825.4 *quinaria,* where Frontinus had 1825. And then he goes on again, as before quoted, worrying himself into all sorts of explanations why his gaugings by areas, made irrespective of heads and velocities, do not balance. The frauds of the water-men, of the plumbers, and of others who draw water unlawfully, always furnish a handy explanation, however.

Another passage relating to the velocity of flow is, 73: "Whence it appears, that, so far from the quantity, which we found, being too great, it should be augmented. The cause of this is to be found in the circumstance that the more torrential stream of water, that is, one taken in from an ample and quick flowing river, increases its measure by reason of this very velocity."

Grants of Water Rights.

But some control of the amounts drawn had to exist, and we accordingly find that the measure of a grant in the city of Rome, A. D. 97, was the right to insert, at a designated place in the public cisterns, which received their supply, either directly or through lead pipes, from the public aqueducts, a circular, bronze ajutage, or short piece of pipe, stamped by the public authority, not less than about 9 inches long and of a designated diameter; some 15 such diameters being in ordinary use; and to allow water naturally to flow through this ajutage; it being the law, moreover, to ensure a natural flow through the stamped bronze ajutage, that the lead or other pipe immediately down-stream from it, should have the same diameter as the ajutage, on a length of not less than 50 feet, measured from the down-stream end of the ajutage.

This is a long definition of a grant; but, nevertheless, did not ensure the draft of a definitely limited quantity of water, as Frontinus himself was well aware. He makes note, for example, that the direction of insertion of the ajutage, relatively to the direction of the current in the

cistern, is of great moment. This is what he says on this point, 36: (An ajutage) "placed at right angles and level maintains its measure; set against the current of the water and sloping down it will devour more; set sloping to one side so that the water flows by, and inclined with the current, that is, placed less favorably for swallowing the water, it will drink without greed, an insignificant quantity."

A mis-quotation, and a mis-translation of the mis-quotation, of a part of this sentence,[1] has been read to mean that a flaring ajutage will increase the discharge; thus attributing a knowledge of the properties of a Venturi tube to the ancients. The words and context prove the falsity of this translation, however.

Bearing on this same matter, and more nearly referring to the properties of a Venturi tube, is the following, 105: "It must, also, not be left optional to attach any kind of lead pipe to the ajutages; but there must rather be attached one of the same interior diameter as that which the ajutage has been certified to have, for a length of 50 feet, as has been decreed by a vote of the Senate, which follows:

The Consuls, Q. Aelius Tubero and Paulus Fabius Maximus, having made a report that some private parties take water directly from the public conduits, have demanded of the Senate what it would please to order upon the subject; upon which it has been ordered: It shall not be permitted to any private party to draw water from the public conduits; and all those to whom the right to draw water shall have been granted, shall draw it from the cisterns, the water commissioners to direct at what points, within and without the city, private parties may erect suitable cisterns for the purpose of drawing water from them; under grants, which in common with others, the water commissioners have located; and no one to whom a right to draw water from the public conduits has been granted shall have the right to use a bigger pipe than a *quinarium* for a space of 50 feet from the cistern out of which he is to draw the water." 112. "In some cisterns, though their ajutages were stamped in conformity to their lawful admeasurements, it was found that pipes of greater diameter were attached to them, which had, as a consequence, that the water not being held together for the lawful distance, and being on the contrary forced through the short restricted distance, easily filled the adjoining larger pipe. Care should therefore be taken, as often as an ajutage is stamped, to stamp also the adjoining pipe over the length prescribed by the vote of the Senate which we have quoted. For thus only is the overseer relieved of every

[1] "Calix devexus amplius rapit."

excuse he could make when he knows that none but stamped pipes are allowed to be set in place." Now the only way to have made a Venturi tube out of such a lawful arrangement of pipes would have been to expand the lead pipe, 50 feet from the cistern, *very gradually* and *uniformly*, into a larger pipe.

Eytelwein made some experiments to test the point, whether a Venturi tube would increase the discharge of a long pipe when placed at the outer end of it. (Gilbert's Annalen, Vol. 7, p. 295.) He experimented with a one inch pipe, 20 feet long, and found that when discharging freely into the air, under a 3 foot head, it made no apparent difference in the discharge, whether the Venturi tube was on or off. This, however, leaves the main question still undetermined, and it remains to be seen whether the same apparatus discharging under water, that is, submerged, or a Venturi mouth-piece, discharging through a long pipe of the diameter of the larger end of the ajutage, will or will not discharge more than the small pipe alone. Venturi thought that it would, and I should not be surprised if it did, to put it no stronger.

Here is another correct conception of the matter of head acting on orifices, had by Frontinus, though he spoke without knowledge of the laws of hydraulics, as they have been developed in the 1800 years following his time. 113. "In setting ajutages, care must be taken to set them on a level, and not place the one higher and the other lower down. The lower one will swallow up more; the higher one will suck in less, because the current of water is drawn in by the lower one."

But the proper placing of the ajutages was evidently kept in the hands of the authorities, and Frontinus had clear ideas on the value of the lead pipes not less than 50 feet long, as we have seen, and that they should be stamped by the public authorities, as well as the bronze ajutage, as a safe-guard against fraud. Two of these ancient bronze ajutages have survived the wreck and plunder of the centuries and are kept, the one in the Kircher, the other in the Vatican Museum at Rome. They have their size and the Procurator's, or owner's, name stamped on them.

If we now frame a commentary on the form of grant above defined, for the purpose of leading to an effective understanding of its provisions, many points may receive notice.

(1) The ajutage must be entered at a designated point. Without this rule it is very evident that the utmost confusion would soon result. Each aqueduct carrying only a limited quantity of water, the right to draw from it, at any point, must be kept in the control of the public authorities, and not be left at the pleasure of owners of water-rights in

general. The Roman water-rights did not "run with the land," but, on the contrary, expired with the decease of the grantee, or his alienation from the plot supplied; or, in case of a syndicate, they expired with the decease, or similar alienation, of the last member of the original syndicate; water-rights thus terminating reverted to the authorities and were granted to other applicants. An exception to this rule was found only in the case of ancient grants for public baths, which were perpetual grants, running with the land (but presumably limited to the purpose named).

From all this, it will be recognized how important it was that the public authorities should determine at what points definite quantities of water should be drawn; that they should have the setting of all ajutages; and should be in position to keep an accurate record of such.

(2) The ajutage must be of a prescribed form, length, diameter and material, and must bear the official stamp of the proper authorities.

Frontinus tells us (24.) how originally, there were square inches and square digits, and round inches and round digits of water-rights in use; until finally, custom settled down to the use of only 15 circular ajutages, varying in size (38.–63.) from the "*quinarium*," $1\frac{1}{4}$ digits (.907 inches English) in diameter, up to a pipe "$12+\frac{4}{12}+\frac{1}{36}$ digits"=(8.965″ English) in diameter; how originally, these were made, apparently, by handing out a plate of lead of a definite width, to be bent or formed into an ajutage; (25.) which method was subject to abuse, by having the lead beaten out thinner, so as to form a tube of greater diameter, until finally, the ajutages were made of bronze, not less than "12 digits" long, and stamped by the "procurator."

Says Frontinus, 36., "Bronze seems to have been selected on account of its hardness; difficult to bend and not easily extended or contracted."

He might have added, or would have, if iron pipes had been known in his day, that bronze alone could be thus used, as it is not subject to rust in running water, and that iron could not be used in place of it on account of its rusting so badly.

Frontinus does not say, but the ajutages were presumably stamped on their interior surfaces, and in several places, and near the ends; or else they could have been bored or filed out larger, or have had the edges flared or rounded off. 112.–115., gives a neat passing picture of some of the frauds readily practiced in ancient Rome, in the matter of an abuse of water-rights, or of thefts of water.

Says Frontinus, 112.: "Having now explained those things that relate to the administration of water for the use of private parties, it will not be foreign to the subject to say something and to give examples

of how we have detected some in the act of contravening the most wholesome ordinances. In a great number of cisterns I found ajutages of a greater size than had been granted, and among them some that had not even been stamped. But whenever a stamped ajutage is larger than its legitimate measure it reveals the soliciting of votes of the *Procurator* who stamped it; but when it is not even stamped, it reveals the fault of all, especially of the grantee, also of the overseer." The rest of 112. is already given on p. 33.

113. "To some pipes no ajutages were attached. Such pipes are called unconstrained, and are enlarged or restricted as pleases the water-men."*

114. "The following method of cheating by the water-men* is, further, unbearable; when a water-right is transferred to a new owner they will insert a new ajutage in the cistern; the old one they leave in the cistern and draw salable water from it. Most especially, as I believe, should therefore the Curator (Superintendent) have in mind, to stop this; for thus he will maintain not only the measure of the water itself, but also the good condition of the cisterns, which get to be leaky when they are so often and unnecessarily tapped into."

Frontinus does not appear to fear a counterfeiting of the official stamp on the ajutages which would be difficult of discovery; and could be discovered only by comparing the granted diameters with the actual diameters; and this means of detection could be used without the use of an official stamp.

Neither does he say anything about, nor was he in a position to appreciate some of the finer points of hydraulic practice, made appreciable by centuries of experience, study and practice since his time. Thus, it would make a difference in the discharge of the ajutage whether the same were tapped into still water, or into a slow or swift current, even though tapped in perpendicularly, as he recommends. The head of the ajutage should, moreover, be exactly smooth and flush with the straight side of the water-course to be tapped, and have sharp square edges. As the level of the water varied in the cisterns, or in the aqueducts, the discharge of the ajutages would necessarily vary with it. This level would vary more at some points than at others, so that owners of the same measure of water-right would not be able to draw the same quantities of water.

Then again, the discharge of an ajutage of 1 inch diameter, is, by no means, proportional to the cross-sectional area of the two ajutages when

* This word meant subordinate public water-works officials.

compared with the discharge of, say a 9 inch ajutage. But Frontinus, as we have seen, compares even the discharge of an aqueduct with the discharge of the several ajutages supplied by it, by comparing the cross-section of the stream of water the aqueduct carries with the sum of the cross-sections of the several ajutages supplied by it. In the light of modern knowledge all this becomes painfully absurd when uttered by so conscientious a writer as Frontinus. But after that, we can pardon, that to him it is all the same, 27. and 28., whether a Roman citizen own 20 rights to insert a "*quinarium*" (a circular ajutage of about .632 square inches, English, area) into a designated cistern, or the right to insert one *vicenarium* (20 *quinaria* in area, about 12.6 square inches, English) in cross-sectional area; though the discharge in these two cases is now known to be materially different.

Arithmetic, A. D. 97, Among the Romans.

One of the astonishing things noted in reading the book of Frontinus is the readiness with which he performs his arithmetical computations, without knowing anything of the Arabic or Indian system of notation, or of common or decimal fractions. To him fractions are entities having names, and represented by hieroglyphics. They have certain relations to unity, and to one another, indeed, but just how he keeps track of them all, and manages to use the lot, is as great a puzzle as the Chinese alphabet; which this set of fractions in some respects resembles.

After the first four of the larger fractions, they are all of the duodecimal order, the list being as follows: $\frac{1}{2}$, $\frac{1}{3}$, $\frac{1}{4}$, $\frac{1}{6}$, $\frac{1}{12}$, $\frac{1}{24}$, $\frac{1}{36}$, $\frac{1}{48}$, $\frac{1}{72}$, $\frac{1}{288}$, with a name for each one. (Semis, Triens, Quadrans, Sextans, Uncia, Semuncia, Duella, Sicilicus, Sextula, Scripulum.) The division by 12 dates back, as is known, to a period of great antiquity, and has come down to us in the 24 hours of the day, and in the 60ths of the hour, and the 60ths of that, as used, together with duo-decimals, by the Greeks; who, according to Herodotus, learnt them of the Babylonians,[1] the 60ths being the modern minutes and seconds, originally called "*partes minutæ primæ*" and "*partes minutæ secundæ*."[2]

The relation of the circumference and the area to the diameter of a circle is given by Frontinus with the use of $\pi =$ as near $3\frac{1}{7}$, $=$ about 3.1429, as the use of the clumsy fractions, just named, selected and

[1] Herodotus II. 109.

[2] Cantor. Gesch. d. Math. I. 445.

added up, will permit of. Working with duo-decimal fractions, it would have been far easier to take $\pi = 3\frac{1}{8}$, as had been done by Vitruvius,[1] only 100 years before Frontinus. But he laboriously plows along with $\pi = 3\frac{1}{7}$, and is always pretty nearly right; except when he, or some copyist, or translator, or printer, has made a mistake; which is not without example.

Thus, he tells us, the *quinarium* has an interior diameter of 1 digit, and three Uncia; or as we would say: 1.25 digits, the digit being $\frac{1}{16}$ of a foot; "in circumference, 3 digits, 11 Unciæ and 3 Scripula;" or as we would say, 3.92709 + digits. Figured on the basis of $\pi = 3\frac{1}{7}$, this figure would be 3.9285 +, and for the ordinarily used value of $\pi = 3.1416$ it is 3.9270.

His areas are $\frac{\pi d^2}{4}$, and he well knows that the areas are as the squares of the diameters. But when it comes to the conception of a cubic foot he seems to avoid it, and he appears to be in total ignorance of so much as a conception of the idea of a procession of such cubic feet, passing a given point in a unit of time, or what we ordinarily call, cubic feet per second; in which respect he is equalled, however, by many a man, some of them of considerable standing in the community, who is living at the present day.

His only idea of quantity is that which passes through an ajutage, of a stamped or certified diameter; by preference, that which passes through a *quinarium*, or as we would say, a "fiver," (about 0.9 of an inch, English, in diameter). One explanation, given by the ancient writers, for this name, was that the pipe was originally made of a strip of lead pipe 5 digits wide. Frontinus ascribes the name to the diameter of the finished pipe, being 5 quarter digits in diameter;[2] and he goes on calling a pipe of six quarter digits in diameter, a *sextarium;* and so on, only that at 20 quarter digits in diameter it becomes nearly the same, whether the name *vicenarium* be ascribed to quarter digits diameter, or to square digits of area. From this size upwards he names the pipes by their areas, in square digits, stopping at a pipe of 120 square digits area, nearly 9 inches, English, in diameter.[3]

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[1] Agrimensoren, 88.

[2] Frontinus, 25.

[3] Pliny: (31.81) and Vitruvius (8.6–4), name the pipes by their circumferences, instead of by their diameters, as is now customary. Rope is sold by circumference measure at the present day.

Quantity of Water Used in Rome, A. D. 97.

What was then the discharge, which Frontinus assumes as a unit? If we suppose, to take the example given by Belgrand, a difference of level of 23 feet, and the point of efflux 670 feet from the cistern, as one reasonable supposition; and only 7 feet difference of level at a distance of 170 feet, for another supposition, according as political or other influence, exerted with the curator, had put a cistern for the consumer, in a more or less favorable situation for the consumer's use; also, supposing 50 feet of *quinarium* pipe at the end of the ajutage, as the law directed; then, ignoring the Eytelwein experiments; I find that if a 15 *quinarium* pipe were carried from the end of the 50 foot length of small pipe, to the point of efflux, either the 23 foot difference of level, or the 7 foot difference, the slopes being nearly the same, (and because the exact discharge depends on the action of the Venturi part,) will yield in the vicinity of 29,000 gallons per 24 hours. But if the *quinarium* pipe were continued all the way, in each case, the discharge would be only 6500 gallons, and 7100 gallons per day, respectively. Other determinations, as will be shown presently, have given the discharge of the average *quinarium* to vary from 6,400 to 13,700 gallons per day, according as they are computed from the data we have, relating to one or another of the 3 aqueducts, whose discharge has been determined with the greatest exactness.

We are now ready to tackle the question of the amount actually brought into Rome daily, by its ancient aqueducts; a question, about which probably more rubbish has been written or repeated, than many another. The chief and original sinner in this regard appears to have been Rondelet, about 1820, the French translator of Frontinus. He calmly assumes, as the basis of his computation, a *quinarium* ajutage, discharging freely into the air, under an assumed head, and ascribes the ridiculous coefficient of 82% to it, into the bargain. Upon which, he figures out about 395 million gallons per day as the discharge of the 9 aqueducts; to be used, or rather consumed, according to the determinations of Latin scholars of the present time, by about 1 million people.[1] This is, evidently, the basis of the Encyclopedia Brittanica figure of 50 million cubic feet, or 375 million gallons per day; found also in many other compilations.

Beck, a German "Privat Docent,"[2] gets his figure of 272 million gal-

[1] Prof. Beloch, the historian, endorsed by Lanciani.

[2] "Der Civil Ingenieur," 1886–619.

lons, simply by assuming 5 feet per second velocity, on the average, in the aqueduct channels.

He calculates the cross-sections of these channels with great difficulty, as Frontinus is exceedingly obscure in his statements in this regard, but vitiates all this work by the unwarranted assumption of 5 feet velocity on the average, in a lot of tolerably rickety masonry channels. I use the word "rickety" as expressing liability to spring leaks and cease to hold water. No such channels could stand 5 feet velocity, and no one with experience in the administration of hydraulic works would assume much more than one-half that velocity, as a *prima facie* estimate. The Roman aqueducts of Nîmes, Lyons, Metz, Antibes and Arcueil, in France, are still standing, and have been much more carefully surveyed than those of Rome. The widths, depths of flow, (from the water-marks,) and slopes are known, and I find their velocities to have been 1 foot, 2 feet, 1.33 feet, 0.2 foot, and 1 foot, respectively.[1] We may conclude, I think, to reject Dr. Beck's estimate of 5 feet per second velocity.

Leger, a French Engineer, and the author hitherto quoted, cannot disenthrall himself from Rondelet's influence, though otherwise a painstaking writer.

He finds 210 million gallons daily, but can not, in my opinion, be adjudged to have sifted his data before commencing to compute.

Belgrand, another, and a noted French engineer, also hitherto quoted, gives us most valuable data respecting the flow at the present day from Marcia, Virgo and Claudia, the waters of all three of which are yet in use. In other respects he also does not seem to be able to emancipate himself from the untrustworthy, more than that, intrinsically false, data given by Rondelet.

It seemed to me, in the course of my studies, that could I get the widths and slopes of the water lines, of the several aqueducts, as they now remain, these two data, taken in conjunction with the *areas* of channel-way, given by Frontinus, would enable the discharge to be computed. More than this: in the case of aqueducts, for which these data could not be got, their discharge could be computed from the discharge of those which had been computed as above, by comparing them on the basis either of *quinaria* given by Frontinus, as found in the channel-way; or of the *quinaria* actually in use, and discharging from the cisterns, as given by Frontinus. I went through all these computations, and the detective work they imply and involve, but came to the conclusion that I would use none of Rondelet's data. His data are unfortu-

[1] Leger.

nately the only ones I could find as giving slopes of the aqueducts. But anybody who would launch upon an unsuspecting public a computation such as above described as having been uttered by him, which includes gross errors, as well as unwarranted assumptions, used as facts, may not be above making other unwarranted assumptions, such as the simple slopes he gives for the Roman aqueducts. But this is a most important factor. It must also be determined with great skill and truthfulness to be of value. Undoubtedly, the slopes of the Roman aqueducts were different, in different parts of the same aqueduct, and the only slope that could be of decisive use to us would be one showing the slope of the *water-surface*, not of the bottom of the channel, and at the points where Frontinus measured his areas of channel-way.

I have accordingly adopted this method: to accept the gaugings of Marcia, Virgo, and Claudia made in modern times,[1] as correctly representing the discharge of the same springs in ancient times. To find from them, what the discharge was, per *quinarium* set and actually drawn from, on the average. The number of such *quinaria*, we have from Frontinus with great accuracy, for he enumerates them for each aqueduct, and then gives the sum total.

In this way I found, that:

Marcia had a discharge per *quinarium* actually in use, in Frontinus' time, of about	13,700	gallons per day.
Virgo " " "	6,400	" " "
Claudia " " "	9,400	" " "
an average of about	9,833	" " "
or in round figures	10,000	" " " .

This figure corresponds, also, fairly well, with the computations made above, on p. 39.

We accordingly have the following table:

Name of Aqueduct.	Quinaria actually set and in use as given by Frontinus.	Discharge in U. S. Gallons per 24 hours.	Remarks.
Appia	704	7,040,000	@ 10,000 gallons per quinarium.
Anio V.	1.610	16,100,000	@ " " " "
Marcia	1,935	26,512,500	Gauging by Blumenstihl about 1870.
Tepula	445	4,450,000	@ 10,000 gallons per quinarium.
Julia	803	8,030,000	@ " " " "
Virgo	2,504	16,012,500	Gauging as above.
Alsietina	392	3.920,000	@ 10,000 gallons per quinarium.
Claudia	2,812	26,512,500	Gauging as above.
Anio N.	2,813	28,130,000	@ 10,000 gallons per quinarium.
	14,018	136,707,000	

[1]Belgrand.

or, some 137 million gallons supplied within and without the city, when the aqueducts were all running. Of this quantity, some 97 million gallons were distributed within, some 40 million gallons without the city. But, we know, from Frontinus' account, as well as from Pliny, that for long periods of time one or more of these aqueducts did not bring water into Rome. For two reasons: the frequent necessity of making repairs upon them, of which I intend to speak later; and because, as the wise it call, their waters were diverted; we will say taken unlawfully, or, perhaps, call it stolen. This is what Pliny says on the subject: (31.25) "And yet for this long time past the pleasure of drinking these waters (Marcia and Virgo) has been lost to the city, owing to the ambition and avarice of certain persons who have turned them out of their course for the supply of their country-seats, and of various places in the suburbs, to the great detriment of the public health."

Taking out Marcia, Julia, (which also was a special prey to thieves,) and Virgo, we have remaining within the city only 50 odd million gallons; to which measure has shrunk Rondelet's unfounded but grandiloquent figure of nearly 400 million gallons per day. 50 million gallons, one day with another, is, however, to the best of my conclusions, a fair estimate, at which to set the water supply within the walls of ancient Rome A. D. 97; ranging, no doubt, 20 or 30 million gallons per day either side of that mark, from time to time. This would make about 50 gallons per day per inhabitant, which is still a very large figure, when use alone, not waste, is taken into account; and considering that by far the greater part of the people undoubtedly used only such water as was carried to their homes in jars on the heads of slave and other women.

Hydraulics After Frontinus' Time.

To appreciate Frontinus' position, with regard to a proper knowledge of the velocity of efflux, and generally of the velocity of running water, it is instructive to follow the development of the art from his time until we arrive at the formula $v = \sqrt{2gh}$ now known to every beginner in hydraulic science, and the very foundation stone of that science, as it is known at the present day.

This formula, and the numerical values it gives to velocities of efflux, was not discovered until about the year 1738, when Daniel Bernouilli and John Bernouilli, his father, each published a different mathematical demonstration of this law. It thus appears that Frontinus wrote

some 1640 years before this fundamental fact was known. By mere number of years in anticipation, he was therefore as much in the dark respecting numerical values for velocity of efflux, as we are, concerning the latest discovery in hydraulics that will have appeared in the year 3534; and even by making due allowance, for the greater rate of speed with which discoveries are now made, compared with ancient times, he was as far in anticipation of the year 1738, as we are of the year, 2300, or 2400. "Better fifty years of Europe," says the poet, "than a cycle of Cathay," and Frontinus' time just preceded a cycle of stagnation, and even of retrogression, compared to which, Cathay may be said to progress with reasonable celerity. For 1200 years after Frontinus, practically no progress was made in the arts and sciences. The first awakenings to a new life, to a revival of learning, may be dated from Roger Bacon, 1214–1293, who preached the importance of experiment, and declared knowledge in his day to be but in its infancy. We who have been educated in English speaking countries, have been accustomed to consider Lord Francis Bacon, 350 years later (1561–1626) as the author and apostle of the experimental method of studying science. But modern research shows him to be entitled to the latter credit, only as he influenced his countrymen of Great Britain, and he himself made no experiments of any note. For a hundred years before his time lived that remarkable painter, sculptor, teacher and engineer, Leonardo da Vinci,[1] (1452–1519,) the misfortune of whose fame it has been that his voluminous works, hidden away for centuries in private keeping, and exposed to manifold vicissitudes, found no publisher until the last few years; and have, even today, not been before the public long enough, to be used by modern writers, as they undoubtedly will be. He not only preached the duty of study by means of experiment, but was himself a most prolific experimenter, and a teacher. In the last named way, he anticipates Lord Bacon; in the other, he is the forerunner even of Galileo. This is what he says on the first named subject:

"In the examination of physical problems I begin by making a few experiments, because it is my desire to state the problem, after I have had the experience of it, and then to show why it is that the bodies are forced to act in the described manner. This is the method it is necessary to follow in all examinations concerning the phenomena of nature. It is true that nature begins, as it were, with argument and ends with experience, but nevertheless, we must follow the contrary way; as I

[1] Grothe, Hermann; L. da Vinci als Ingenieur, &c., Berlin, 1874.

have said, we must commence with experience, and strive by means of it to discover truth."

"In the study of the sciences which are allied to mathematics, those who do not consult nature, or authors who are not the pupils of nature, are merely little children. I say it emphatically. Nature alone is the true teacher of true ability. And yet, behold the stupidity of it! The world makes merry over a man who prefers to learn from nature rather than from writers; who themselves could only be the pupils of nature."

His experiment on the law of falling bodies is most interesting in connection with the matter we are now considering.[1] He used two long boards hinged together like the leaves of a book. On the inside these boards were smeared with tar or wax. A string latch served to suddenly close them. He then takes a small tube, filled with shot, the tube having nearly the same diameter as the shot. This tube is held vertically in and over the angle of the wooden book, itself set up vertically. The shot are then allowed to drop out, and on pulling the latch, are caught as they fall between the leaves of the wooden book, and their relative distances, as they are falling, are impressed on the tar or wax covering of the boards.

Until quite recently Galileo has been supposed to have been the first experimenter on the laws of falling bodies, but here was this great engineer and teacher busily at work at it 100 years previously. However, with Galileo (1564–1642) we first touch the science of "dynamics," or of bodies in motion. Says Rühlman:[2] "For the proper founding of the science of dynamics, or of the science which treats of the causes and the laws of motion, were requisite talents of a degree of eminence, such as the Lord Almighty called into being with Galileo in the year 1564."[3] But Galileo had no proper means for measuring time, no clocks or watches. Both he and his son tried to make a clock but did not succeed. Instead, he used a large bowl of water, having a small orifice at the bottom, and compared times by the weights of water discharging during these times; using his finger to start and stop the flow of water out of the bowl. As we shall see, it is a reasonable assumption that this makeshift of a clock became, in the hands of Galileo's pupils, and in those of his pupil's pupil, the suggestion for an experimental demonstration of the laws of efflux in general.

[1] da Vinci, Leonardo. Del Moto e. Misura dell Acqua, Bologna. 1828. p. 364.

[2] Gesch. d. technischen Mechanik, p. 53.

[3] Libri, Histoire des Sciences Mathematiques, IV, p. 160, 466, shows that Lord Francis Bacon knew Galileo's works, published and unpublished, a year before the publication of the "Novum Organon."

Castelli (1577–1644), the pupil of Galileo, was a Benedictine monk, from that same Monte Casino which saved Frontinus' commentary to posterity, and he first showed that the quantity of efflux, in a given time, depended, by law on, or was a function of, the depth of water in a bowl, such as the one just spoken of; that is, was a function of the head. But he wrongly stated this law; making the quantity vary directly as the head. It was his pupil, Toricelli (1608–1647), the inventor of the Barometer, the grandson, in a professional sense, of Galileo, who first proved, in 1644 or only 2 years after Galileo's death, that the velocities of efflux are as the square roots of the head. But this still furnished no numerical value for the velocity of efflux. Still another, and still other great men, had to devote their lives to this cause; 30 more years had to pass by, till Huygens (1629–1695) the inventor of pendulum clocks, first found the numerical value of the acceleration of gravity, commonly represented by the letter g, in 1673; and 65 more years had to elapse, until the genius of the two Bernouillis, father and son, in 1738, or 250 years after Leonardo da Vinci, finally laid the foundation of modern determinate hydraulics, by writing the equation of $v=\sqrt{2gh}$, every letter and character of which may be considered the contribution of, and a tribute to the skill and perseverance of one or more of the many great men I have named. v. may stand to symbolize the experiments of da Vinci and of Galileo, and the preaching of the two Bacons; $2g$. alone would suffice to immortalize Huygens, were he not already permanently distinguished by his invention of pendulum clocks and other works; h. may serve to re-call Castelli; and the square root sign, his pupil, Toricelli; and when next we write the formula, let us remember that it took 250 years of work, not to speak of another and a preceding 250 years, or more, of speculation, to put it upon the blackboard of the world. But no amount of speculation alone, or of peripatetic philosophy, would have produced it. To do that, the work of centuries of earnest men, not too proud to dip their hands into bowls of water, and to *experiment* in hydraulics, the while they were wearing mechanic's over-alls, so to speak, was absolutely necessary.

No wonder then that Frontinus, who lived in the age of peripatetic philosophy, did no better than we have found in his II Books, in the way of gauging the water supply of ancient Rome.

Frontinus as a Superintendent of Water Works.

But whatever Frontinus may have lacked in knowledge of the laws

of hydraulics, certain it is that he takes high rank, even to-day, as an administrator, or practical Superintendent of water-works. His own language, describing the operation of the Roman water-works, is the best illustration of this.

9. "Near the intake of Julia is a brook called 'Crabra.' Agrippa disdained to take in this brook, be it because he did not consider it of good quality, or because he thought he was under obligations to leave it for the use of the proprietors of the gardens at Tusculum; for it is the same, which is distributed by turns, on fixed days, and in determined quantities, to the estates of that part of the country. But without the same moderation, our water-men constantly drew upon the greater part of it for the replenishment of the Julian aqueduct, though not for the purpose of increasing the flow of this aqueduct, which, on the contrary, they exhausted, by diverting its waters to their own profit. I therefore cut off the Crabra brook, and gave it again, upon the orders of the Emperor, to them of Tusculum, who perhaps now get it with great astonishment on their part, and without knowing to what cause they may ascribe the unusual surplus. The Julian aqueduct, on the other hand, has regained its normal quantity even during notable droughts, by reason of the destruction of the branch pipes, through which it was secretly despoiled."

87. "This is the schedule of the amounts of water distributed or available, down to the time of the Emperor Nerva. But now, has all that been recovered by the painstaking of the most diligent of sovereigns, which had been unlawfully drawn by the water-men, or had wasted by means of official negligence, this being equivalent to the finding of new sources of supply, by means of which an abundance has been procured. Afterwards a more rational distribution was undertaken, so that those wards which were supplied by only one aqueduct, should receive the waters of several;" and so on, showing that this was done to keep all the wards supplied, no matter which particular aqueduct might, for the time being, be drawn for purposes of cleansing or repairs. For the same reasons, also, the running fountains were connected by lead pipes, each one with generally at least two cisterns, the two cisterns being supplied by different aqueducts.

88. "This care of Nerva, the gentlest of Emperors, her ruler, experiences from day to day, the present queen and empress of the world; and still more will experience it, the health of the same, the Eternal City, after increase in numbers of the reservoirs, cisterns, fountains, and the water basins. No little advantage accrues also to private con-

sumers from the increase in number of private grants; and those who with fear drew water unlawfully, draw their supply now securely by grants from the sovereign. Not even the waste water is idle; wholly different is now the cleanliness of the city; the air is purer; and the causes of the nearly suffocating exhalations, which gave the air of the city so bad a name with the ancients, are now removed."

89, 90, 91, 92, 93, treat of a separation of the waters of the several aqueducts, in the uses to which they were put, by allotting them, with reference to the qualities of each water. Some waters were mixed, by reaching the city for part of the distance, in one and the same aqueduct channel, "so that we have found even Marcia, so charming in its purity and coldness, used for baths, fulling mills, and I may not say what vile appointments."

98. "At first Agrippa (B. C. 34) was perpetual *curator*, as it were, over his public works and ornamental fountains, after having been aedile, which was after he had been consul, and he apportioned the quantity of water then flowing; allotting so much to the public structures, so much to the basins, and so much to private parties. He also kept a family of slaves for the maintenance of the aqueducts, and cisterns, and basins. This family was given to the state as its property by Augustus, who had received them in inheritance from Agrippa." Augustus did a great deal to organize the administration of the works, and appointed (about 22 B. C.) Messala Corvinus the first *curator*.

Of these slave bands there were two; the one already mentioned, numbering 240 men, called those of the State; and another, called those of Cæsar, founded by Claudius (about 40 A. D.) numbering 460 men. Many trades were represented among them, such as overseers, reservoir-keepers, line-walkers, pavers, plasterers, plumbers, masons, &c.

117. "Of these, some must be outside the city, for purposes which do not require any great amount of work, but require to be seen to promptly; the men within the city at their stations at the reservoirs and fountains, must strive to do all work, especially in cases of sudden and unexpected events, so that a more plentiful supply of water may be turned from several wards of the city to those who are threatened with a drought. The whole of both families of slaves, who occasionally were taken, by exercise of favoritism or negligence of their superiors, for the use of private work, we resolved to bring back to some discipline, and for the service of the State, by writing down the day before what each one was to do, and by putting in the records what they did each day."

118. The water-rights, granted to gardens and buildings, adjacent

to the aqueducts or cisterns, or fountains, or water-basins, yielded nearly 250,000 sestertia annual income, about $10,000 worth weight of gold or $40,000 of money to-day.

120. "Work of maintenance arises from the following reasons: by lawlessness of the owners of fields traversed; by age; by the weather or by poor workmanship in the original construction, which happens frequently in the case of recent work."

121. "As a rule, those parts of the aqueducts, which are carried on arches, or are placed on side-hills, and of those on arches, the parts that cross rivers, suffer most from the effects of age or of the elements. Therefore must these be built with careful diligence. The underground portions, not being subjected to either heat or frost, are less liable to injury. Repairs are either of the sort that can be made without stopping the flow of the water, or such that cannot be made without emptying the conduit, as are those to be made in the channel itself."

122. "These arise from two causes: either by increase of deposit, sometimes hardening in form of a crust, and thus diminishing the size of the channel; or, by destruction of the concrete lining, causing leaks, which of course do injury to the side walls of the channel, and to the substructure. Sometimes even the piers which are built of 'tufa' yield under their great load. Repairs to the sides of the channel should not be made in the summer time, so as not to stop the flow of water at a time when the demand for it is the greatest; but should be made in the Spring or Autumn, and, moreover, with the greatest speed possible, so that, all preparations for hurrying the work having first been made, the flow of water may be interrupted as few days as possible. As everyone can see, one aqueduct must be taken at a time, for if several were drained, a lack of water to the citizens might arise."

123. "Repairs that can be carried on without drawing the aqueducts consist principally of masonry, which should be laid up at the right time and conscientiously. The proper time for masonry is, from the calends of April to the calends of November; but, as is best, in such a way, that that portion of the summer during which too great a heat aglows, may pass without work going on; because moderate weather is necessary for the masonry to suck in the mortar, and to solidify uniformly. But no less than the heat of the sun, does too violent frost act prematurely destructively upon masonry; and yet is no greater care required upon any work, than upon such as are to withstand the action of water."

124. "But even for these difficulties (to repair a conduit on arches,

without stopping the flow of water any great length of time) are there methods of relief; a foundation is built up to the level of the defective conduit, and the channel is lined over the length of the stripped portion, with lead."

128. "But many (land-owners) have not been content to assume control beyond the boundaries, but have laid hands on the aqueducts, by diverting, here and there, some of the water to their own use through the side walls of the punctured channels; this being done, not only by those who have a right to draw water, but also by those who misuse the least favor given them, by attacking the walls of the conduits."

One of these applications for a grant of water for private uses has come down to us. It is written in verse and is by the poet Martial, who was constantly currying favor with, or begging something of somebody. I translate his plea as follows: (Book 9. Epigram 18)

TO CÆSAR DOMITIAN.

I own, Cæsar, and may it long be under thy reign, a little villa, and a modest city hearth-stone. But it is by force of arms, and by means of a chain-pump that we draw from a narrow well, the water needed by my suffering gardens; thus it happens, that while my domicile complains of not receiving a drop of moisture, I yet hear the murmur of my neighbor Marcia's stream. The permit that thou woulds't grant to my house-hold, Augustus, will be to it like as the waters of Castalia, or as the showers of Jupiter.[1]

Let us hope, for the sake of the rest of the Roman public, that this petition was referred to a superintendent, as keen in sense of duty as was Frontinus, and with power to act; and if so, we may be sure that Martial received no water for irrigation purposes, at the expense of those who needed it to drink.

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[1] AD CÆSAREM DOMITIANUM.

Est mihi, sitque precor longum te præside, Cæsar,
  Rus mininum; parvi sunt et in urbe lares,
Sed de valle brevi, quas det sitientibus hortis,
  Curva laborates Antlia tollit aquas.
Sicca domus quæritur nullo se rore foveri,
  Quum mihi vicino Martia fonte sonet.
Quam dederis nostris, Auguste. penatibus undam
  Castalis hæc nobis, aut Jovis imber erit.

MAINTENANCE AND REPAIRS.

A review of the ancient inscriptions, and other sources of knowledge, that relate to the repairs constantly made necessary on the aqueducts of Rome, inevitably brings us to the conclusion, that so far from meriting unalloyed admiration, or that they be taken as models, at the present day, as regards their fitness to convey water, and their durability of construction, they may be subjected to valid engineering criticism, both as to methods of construction, and of operation. Frontinus recites how Appia and Anio Vetus were out of repair, when Marcia was built (145 B. C.); how Agrippa (34 B. C.) built Julia, and *in the same year*,[1] restored Appia, Anio Vetus, and Marcia; that is to say, that none of these aqueducts could have been out of repair over any great length; or else they could not all have been restored to use in one year, in which, moreover, a new aqueduct was under construction. Nevertheless, only 29 years later, as we read on the St. Laurentian gate, "Augustus, son of the divine Cæsar, sovereign Pontiff, in the 12th year of his consulate, the 19th year of his office as tribune, 14 times proclaimed Emperor, restored all the water channels." "Rivos aquarum omnium restituit." This was 5 B. C. But other inscriptions tell of repairs on Virgo A. D. 31, again in 43, and in 44; on Claudia and Anio novus in 52, Claudia again in 71, after 9 years of disuse or after running only 10 years; Marcia in 79; Claudia and Anio novus again in 80 after running 9 years; and Claudia again in 84; then Marcia again in 103, being about the time of Frontinus' death. It is very evident that these works suffered severely from the action of the elements; and it may be argued, from lack of efficient care.

In the first place, it is all wrong to construct of masonry, any continuous, simple, channel above ground, and expect it long to hold water. The mere expansion and contraction of the stone-work, beaten upon by the great heat of the sun in summer, as Frontinus tells us, and exposed to frost in the winter, will speedily crack such masonry channels, or their thin concrete lining. A crack once formed, the water will soon make it larger, for to no work is the proverb of "a stitch in time, saves nine" more applicable than to works conveying or storing water. Again, those families of slaves we read about, as having charge of the aqueducts, were no doubt a shiftless set, as slaves were or are everywhere; from ancient times, and in ancient countries, down to times that some of us can remember, and in places nearer home, that, thank God, know that cause and degree of shiftlessness no longer.

[1] Frontinus, 9.

The ruins of the aqueducts still existing give ample proof of the difficulties encountered in maintaining them water-tight, and I shall later show you, by photograph, one method that was used by the Romans in repairing leaks.

The cupidity and thoughtless selfishness of the public, also, contributed in no small degree, as we have seen, to the diversion of the waters of the aqueducts before they reached the city. To cap the climax, the right to take and conduct water leaking from the aqueducts, or overflowing from fountains, or otherwise wasting, was made the subject of a grant. It requires but little imagination to see how such grants were not suffered to lapse, by the grantee, for want of water wasting or leaking, if he could help it; and he, no doubt, generally could. Leaks probably grew larger, instead of being promptly repaired; and overflow and waste could be readily increased by the assistance, or only passive demeanor, of a properly persuaded water-man.

We may remember, in this connection, that description of Fabretti's, of the lime deposits, from leaks that formed in places on the outside of the aqueducts, wherein he likens them to hay-stacks. Leaks of this sort evidently ran both a long time and in abundant quantity; though this would not necessarily prove shiftlessness of management, except at a comparatively recent period. Considerations, such as these, readily explain to us, how these frequent overhaulings and repairs of the aqueducts were made necessary; and how in spite of them, as we have seen, Rome seldom, if ever, received the water of all her 9 aqueducts at the same time.

To continue the history of these 9 aqueducts, we find Caracalla repairing them all in 212. Claudia is running in 399, and in 402, as shown by two laws then passed with regard to Claudia. In the year 500, Theodoric writes to Cassiodorus about Virgo, as already quoted. In 536, Procopius coming to Rome with Belisarius, finds 11 aqueducts there.

In 537 Rome is besieged by the Goths and Burgundians, who destroy the aqueducts. They made a fort, or walled camp, of one portion of them; a tower thus built by them, at the intersection of several of the aqueducts, standing to this day. Nevertheless, Belisarius restored Claudia and Trajana between 537 and 549. Somewhere between 548 and 568 the aqueducts all ceased to convey water, and so remained until about 776, or over 200 years, when the Popes, put in power by Charlemagne, commenced to repair them.

In 776 Adrian I. restored Trajana, Marcia, Claudia and Virgo, in the order named.

We find Claudia in use in 795, under Leo III., Adrian's successor. From 1120 to 1122, Calixtus II. used Claudia, and that is the last we hear of Claudia as a running stream of water; after its fitful existence of 1070 years.

Virgo was restored by Nicholas V. in 1447, and remained in use thirty years. Seven years after, 1484, Sixtus IV. restored Virgo. 1550 to 1555 Julius III. used its leakage waters, and in 1559 it had ceased to flow. That year Pius IV. repaired Virgo, which repairs sufficed for eleven years. Then in 1570 Pius V. restored Virgo; since when Gregory XIII., Clement XII., in 1735; and Benedictus XIV. in 1744, made changes, extensions and improvements; and the spring "Virgo" is conveyed to Rome to-day, and yields some 16 million gallons per twenty-four hours.

THE LAW OF WATER-RIGHTS, IN ROME, A. D. 97.

Another instructive feature of the II Books of Frontinus, are the paragraphs in which he deals with the law of water-rights, a subject that now, as well as then, no conscientious supervisor or administrator of water-power, or of water-conducting properties, can afford to ignore or neglect. It is significant to observe, in this connection, that in Bruns *Fontes*, a work giving the sources of the Roman law, the subject of water-rights is nigh excluded, except as it is given by extracts from the essay of Frontinus. I commence with 94. "No private person shall conduct other water, than that which flows from the basins onto the ground." This is ancient law, says Frontinus. And even this much was granted only to public baths and to fulling mills. In another place 110. Frontinus adds: "Those waters also that we have called lapsed, namely those that come from leakage out of the cisterns or the pipes are subject to grants; but which are but rarely given by the sovereign."

97. "Fields, of which it could be proven that they had been irrigated unlawfully from the public supply, were confiscated to the State. A tenant who allowed a slave of his to act against the law, was fined." Another law read thus: 97. "No one may with malice pollute the waters where they issue publicly; should any one pollute them his fine shall be 10,000 sestertia" (equal to $400, or say $1,000).

103. "No one may draw water without a writing from Cæsar, that is, that no one may draw water from the public supply without a written license, and no one more than has been granted." "By this means we propose to make it possible," says Frontinus, "that the quantity of

water, concerning whose recovery we have spoken, may be distributed to new fountains, and for new grants from the sovereign. But in both cases must a great zeal in the service be exposed to manifold forms of fraud. The channels of the aqueducts, without the city, must be frequently examined, one after the other, to locate the granted quantities; the same must be done in case of the cisterns and fountains, that the water may flow without interruption day and night; which the curator has been directed to see to by vote of the Senate, whose words are as follows."

Follows a long vote of the Senate, 104, thus specifying. Another law: 105. "Whoever wishes to draw water for private use must seek for a grant, and bring to the curator, a writing from the sovereign."

106. is the law prescribing 50 feet of pipe next the ajutages, already spoken of. Also prescribing the draft to take place from the *cisterns*, so that the public pipes be not tapped too often.

107. "The right to granted water does not pass either to the heirs, nor to the buyer, nor to any new occupant of the land. The public bathing establishments had from old times the privilege, that water once granted to them, should remain theirs forever. We know this from old votes of the Senate, of which I give one below. Now-a-days, every grant of water is renewed to the new owner." And then follows the law, 108, which I will not transcribe.

109. Thirty days notice was ordered given by the Emperor, Nerva, before the water was to be shut off. In case of syndicates, the grant held till the last member of the syndicate alienated his interest in the estate.

"That granted water cannot be carried elsewhere than upon the premises to which it has been made appurtenant," says Frontinus, "or taken from another cistern than the one designated in the writing of the sovereign, is self-evident, but is also forbidden by ordinance."

125. treats of the subject of taking by right of eminent domain materials from adjoining estates, for the purpose of making repairs, and the taking of rights of way to the aqueducts, for the same purpose. They are appraised by an arbitrator, and paid for.

127. gives the full text of the law, of 11 B. C., forbidding, under penalty of a fine, the planting of trees or otherwise occupying a strip, 15 feet wide, each side of the aqueducts.

128. touches upon the right of eminent domain for the taking of lands. In constructing the works, the predecessors of Frontinus had

often taken the whole of a man's estate, whenever the owner so preferred, had then used what was necessary, and had sold the balance.

129. gives the full text of the Lex Quinctia (39 B. C.), which forbad, under a penalty of 100,000 sestertia ($4000, or say $10,000), the malicious destruction of any part of the aqueducts, or the unlawful draft of water from them. And we may be sure that during Frontinus' administration some use at least was made of this law.

An interesting legal exhibit, relating to Frontinus' time, has been lost to us. When it is considered what little interest 1000 years of monkdom presumably had in temporal subjects, in practical hydraulics for example, the wonder is not that so much has been lost, but rather, that so much was nevertheless preserved. It is well to remember that our knowledge of ancient times is not entirely such as the ancients themselves pictured themselves, but only such as has been retained on the filter of monkishly transcribed manuscripts. According as a monk took an interest in ennobling, or in trivial or even in debasing subjects, he transcribed and circulated the one or the other class of literature; but none of them, in the first 10 centuries of our era, probably cared much about water, for either domestic, navigation, power, or even ablutionary purposes.

Says Frontinus: 76. "Concerning misdemeanors of this sort (unlawfully taking water) nothing better or additional can be said than was said by Cælius Rufus in his speech, which is entitled 'Concerning Waters.' And we could prove that all these misdeeds are still being committed, were it possible to enumerate them without vexation of spirit: such as irrigated fields; taverns; also dining-rooms; lastly, disorderly houses, have we found fitted up with constantly flowing fixtures. For that some waters should be delivered without right of title, or some waters in place of other waters, belongs to the lesser misdemeanors."

This last sentence is one of profound wisdom, as will be recognized by all who have ever had experience in the practical administration of water-supply, or of water-power properties. In fact, the reading of many another part of Frontinus' little essay of 50 pages, will afford but additional proof, how like the ancient times and manners, are those of modern date. From fondness for the essay, one comes imperceptibly to have fondness for the writer; this conscientious, honest old Roman water commissioner; and to regret, as we may now do in closing, that no more of his writings have been preserved to us.

Notes on Certain Field Methods Used on the Survey of the Mexican Boundary in 1892-3.

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To the Civil Engineering Association of Cornell University:

GENTLEMEN: The following paper is in the main a description of certain methods used by the author, on the survey of the boundary between the United States and Mexico from El Paso, Tex., westward to San Diego, Cal., in 1892–3.

The Commissioners on the part of the United States in charge of this survey were Lieut.-Col. J. W. Barlow, Corps of Engineers, First Lieut. D. D. Gaillard, Corps of Engineers, and Mr. A. T. Mosman of the Coast and Geodetic Survey.

Though the methods in question are much like the orthodox, well-tried methods, yet they involve some important changes in the usual practice,—changes that arose from the study in the field of problems actually encountered. A belief that progress is made by holding to every result of experience, even though it cause but apparently small improvements, furnishes the motive for the presentation of this paper.

The paper deals with the determinations of latitude as made on this survey, the azimuth determinations, a method used in tracing long, straight lines upon the ground, and the action of heliotropes as used on this survey under conditions which are usually considered abnormal.

All the latitude observations at the fifteen astronomical stations of the survey were made with the Würdemann zenith telescope No. 20. This instrument was formerly used on the survey of the northern boundary from the Lake of the Woods to the Rocky Mountains. It was thoroughly cleaned at Washington, D. C., by Fauth & Co., in December, 1891. The micrometer was then remodeled and the old micrometer screw replaced by a new one of about the same pitch,—one hundred

threads to the inch. The latitude levels were also replaced by new ones at that time.

The telescope had a focal length of 826^{mm}, the objective had a clear diameter of 67^{mm}, and the eyepiece magnified about seventy diameters.

The latitude level carried a 2^{mm} graduation of 70 divisions *numbered continuously from one end to the other.* The marked advantage of this method of numbering the divisions is, that an exchange of north and south readings of the level can readily be detected by the recorder or from the record. The value of one division of level was $1''.28$. The level tube was protected from sudden changes of temperature by an outer glass tube.

The clamp for vertical motion of the telescope was an axis clamp,—probably not the best form of clamp for this purpose.

The instrument was usually mounted on a wooden pier in the large observatory tent, which also contained the azimuth instrument mounted ready for use, and which served as an office during the day. The floor space of this tent was nine feet from north to south and twelve feet from east to west, and its shape that of a "lean-to" shed, with roof sloping downward toward the north. The sides and roof were securely supported throughout by a wooden framework. The floor was made so as to be portable, in six sections, and was so supported by sleepers that no pressure was transmitted to the ground, either from the weight of the tent or of its occupants, except at the four corners of the tent,—as far as possible from the piers.

The latitude pier was a hollow triangular column built of two thicknesses of three-inch pine put together with screws and banded at top and bottom with heavy hoop iron. The foot-plates of the instrument were screwed directly to the end of the column without the intervention of any cap. The pier was set at each station like a fence post,—about 60^{cm} of its length being below ground. The earth was tamped solidly around it and its hollow interior was also filled with earth to give it greater firmness. The stability of this wooden pier seemed to be as great as that of a brick or stone cemented pier and its use saved much time and transportation in a country where neither cement nor water could usually be obtained without being hauled a considerable distance by wagon.

The readings of the latitude level gave a sensitive test of the stability of the pier. The instrument, after having been leveled, usually remained for two or three hours with its vertical axis so nearly vertical that the level correction was less than one division, namely, $1''.28$. It was not uncommon for the level correction to remain without releveling

less than one division during the whole of a night's work, even in cases in which the observations extended through nearly all the hours of darkness. The observations for a night usually show a very slow motion of the vertical axis of the instrument so as to incline more and more to the southward,—as if the southern side of the pier were becoming gradually shorter relatively to the northern side. This motion was exceedingly slow,—so slow that, as stated above, the level correction usually remained less than one division for hours at a time without releveling.

The door of the tent was usually kept open during observations, and the circulation of air through the door sides and roof of the tent generally kept the temperature inside the tent within a degree Centigrade of the temperature outside. This seemed to be an important factor in controlling the apparent steadiness of the stars, for, at Station No. 12, with the instrument mounted in a wooden building, and at No. 13 in an adobe building, with the outside meteorological conditions just as favorable as before, there was a much greater difference between inside and outside temperature, and the stars showed a marked unsteadiness as compared with their usual appearance.

Most of the observations were made under conditions very favorable to accuracy. The typical night of observation was perfectly clear, with no strong winds, and with air so dry that no dew fell even during the cool morning hours. Under such conditions the stars, except near sunset, showed very brightly, with little or no twinkling or dancing, as seen with the telescope.

The value of micrometer was determined at every station, save one at which only a second-class latitude determination was required. The list of pairs observed was so selected as to eliminate as nearly as possible the effect of the error in micrometer value upon the final computed value of the latitude. In balancing micrometer differences *the actual observed micrometer differences were used and account was taken of the effect of the different weights assigned to different pairs.*

The following table indicates the degree of accuracy attained at each station and the plan of work as to number of observations and number of pairs:

Station.	No. Obs.	No. Pairs.	e	e_δ	e_ϕ	Total Range.	Greatest Range on Any Pair.	No. Obs. Rejected.
No. 1	67	19	±0".33	±0".15	±0".06	3".28	2".10	1
No. 2	39	39			±0.07	3.29		
No. 3	46	46			±0.05	2.33		1
No. 4	130	19	±0.36	±0.09	±0.04	4.04	3.08	1
No. 5	99	50	±0.22	±0.21	±0.04	2.04	0.95	
No. 6	103	57	±0.20	±0.24	±0.04	2.28	1.26	
No. 7	99	63	±0.26	±0.20	±0.04	2.50	1.76	1
No. 8	100	75	±0.21	±0.18	±0.03	2.02	0.95	
No. 9	101	72	±0.19	±0.24	±0.03	2.21	0.88	
No. 10	106	76	±0.21	±0.25	±0.04	2.18	0.99	
No. 11	126	92	±0.23	±0.14	±0.03	2.08	1.06	
No. 12	121	93	±0.31	±0.29	±0.04	3.66	1.45	
No. 13	105	45	±0.28	±0.02 ?	±0.03	2.35	1.40	
No. 14	25	24			±0.03	2.30		
No. 15	96	76	±0.22	±0.32	±0.04	2.95	1.26	3
	Sum, 1362							Sum, 7

The number of observations as given in the first column is the number actually used after the rejection of erroneous observations as shown in the last column.

e is the probable error of a single observation.

e_δ is the probable error of the mean of two star declinations.

e_ϕ is the probable error of the final value for the latitude of the station.

The computations were made by the usual formulae as shown in Appendix No. 14 of the Coast and Geodetic Survey Report for 1880.

At Stations No. 2 and No. 3 where each pair was observed but once, and at Station No. 14 where only one pair was observed more than once, the probable error of a single observation, e, cannot be derived by the ordinary formulae as referred to above. In those cases the ordinary formulae for independent observations of equal weight were used to determine the probable error of the mean latitude, and the probable error of the result for latitude from any one pair,—including *both* errors of observation and of declination. The probable error of the latitude from a single pair observed once as computed thus is for Station No. 2 ± 0".44, for Station No. 3 ± 0".35, and for Station No. 14 ± 0".38.

The computed value for the probable error of a single observation depends only upon observations on pairs which were observed more than once. For Stations No. 6 to No. 12 and No. 15, it therefore depends upon but a small portion of the observations. As the computed probable error of declination, however, depends upon the difference between

e_δ^2 (a quantity computed from *all* the observations) and e^2, too small a value for e will give too large a value for e_δ and vice versa. For this reason the two columns giving e and e_δ should *both* be considered in judging of the accuracy of the observations.

As the general plan of observation adopted at most of the stations of this survey differs materially in one important respect from that usually followed in zenith telescope latitude determinations, it is pertinent here to point out the reasons which led to the change from the usual practice and to note how the results have justified the change.

The ordinary procedure at a first-class zenith telescope latitude station is to observe the *same* list of about twenty pairs on from four to seven nights. That plan was followed at Stations No. 1 and No. 4. Starting with the premise that the prime object of the latitude observations is to secure with a minimum expenditure of time and money, a result for latitude having a given probable error, say $\pm\ 0''.05$ or less, a study of the relative magnitude of the errors arising from various sources and the cost of reducing those errors, led to the conclusion that the number of independent pairs observed should be greatly increased relatively to the number of observations. It seemed that the greatest accuracy for a given amount of money would be secured by observing each pair but once, or at most twice. Accordingly the method of securing as many independent pairs as possible was followed at all stations except No. 1 and No. 4. The results of this procedure are its best champion.

A simple comparison of the probable errors of the final results at Stations No. 1 and No. 4 with the corresponding probable errors at later stations is not fair to the old plan of work. For at Stations No. 1 and No. 4 unfavorable conditions, namely, inexperience of observer at No. 1 and bad meteorological conditions at No. 4, made the probable error of observation greater than at any succeeding station.

At Stations No. 2 and No. 3, however, the probable error of a single observation was about the same as at No. 1 and a comparison is just. At No. 2 and No. 3,—39 and 46 observations respectively, no pair being observed more than once, gave about the same degree of accuracy in the final result as 67 observations at No. 1 made on the old plan, each pair being observed three or four times.

At Stations No. 5 to No. 12, where the plan of work was most uniform, from 99 to 126 observations were taken at each station on from 50 to 93 pairs, no pair being observed more than twice. The probable error of the final result for each of these stations is either $\pm\ 0''.04$ or $\pm\ 0''.03$. Taking the average values for these eight stations for the number of observations and of pairs, for e and e_δ, there is obtained as a

typical station 108 observations on 72 pairs with $e = \pm\ 0''.23$ and $e_\delta = \pm\ 0''.22$. For this typical station the probable error of the final result is

$$\left[\frac{(0.23)^2}{108} + \frac{(0.22)^2}{72}\right]^{\frac{1}{2}} = \left[(0.022)^2 + (0.026)^2\right]^{\frac{1}{2}} = \pm\ 0''.034.$$

If the same number of observations were taken upon 18 pairs, 6 observations per pair, e and e_δ remaining the same, the probable error of the final result would be

$$\left[\frac{(0.23)^2}{108} + \frac{(0.22)^2}{18}\right]^{\frac{1}{2}} = \left[(0.022)^2 + (0.052)^2\right]^{\frac{1}{2}} = \pm\ 0''.056.$$

An *infinite* number of observations on 18 such pairs would give a result with a probable error of

$$\frac{0.22}{(18)^{\frac{1}{2}}} = 0''.052,$$

that being the error from declination alone. Even if the probable error of a single observation were twice as great as in the actual case, namely $\pm\ 0''.46$, and e_δ were $\pm\ 0''.22$ as before, 108 observations on 72 pairs would give a result with a probable error of

$$\left[\frac{(0.46)^2}{108} + \frac{(0.22)^2}{72}\right]^{\frac{1}{2}} = \left[(0.044)^2 + (0.026)\right]^{\frac{1}{2}} = \pm\ 0''.051,$$

— a more accurate result than could possibly be obtained from 18 such pairs observed any number of times with any degree of accuracy.

In cases in which the accuracy of the available star places is not as great as that indicated by the above value for e_δ there is still greater advantage in the method of observing a large number of pairs.

This method of work involves the necessity of computing a greater number of mean star places than usual. But the cost of that portion of the work is only a small part of the total cost of the latitude determinations. The cost of preparing the whole list of about 620 accurate mean star places used on this survey at 15 stations was about the same as the expense of maintaining the astronomical party in the field for a single fortnight. A considerable portion of this list would have been needed even if the old plan of work had been followed.

The cost of the remainder of the computation is substantially the same for a given number of observations whether they are made upon many pairs or a few pairs only.

It may truthfully be said that when few or no pairs are observed more than once that the probable error of observation cannot be satisfactorily determined from the observations. But the latitude observations are made for the purpose of obtaining an accurate determination

of the latitude of a certain point rather than to test the skill of the observer. Observations on *many* pairs give just as good a determination of the probable error of the *final* result as repeated observations on a *few* pairs, but they will not indicate so clearly what relative proportions of that error are due to the observing and to the star places. Is it so important to determine the relative magnitude of the errors from these two sources that accuracy in the final result should be sacrificed for that purpose?

So, too, it is true that in the common case in which the party in the field is not furnished with *accurate* star places, the observer can form a good estimate of the reliability of his work if each pair is repeated several times, by comparing the results from the same pair with each other; whereas, if each pair is observed but once, he will not be able to know whether large residuals are due to defective star places or to poor observing. But is it desirable that the observer should *certainly* increase considerably the inaccuracy of the *final result* (by neglecting to avail himself of additional pairs) for the purpose of being a little more certain while in the field as to whether two or three observations which appear doubtful should be rejected?

An incidental, but important, advantage of the method of observing *many* pairs is the way in which partially cloudy nights and isolated clear nights in the midst of a cloudy season, may be utilized. If one is observing in the ordinary way on a list of say twenty pairs, only three or four hours of a perfectly clear night can be utilized, even though there may be great anxiety to finish the work at the station. On the other hand, observations for the night are entirely prevented if it is cloudy during the three hours covered by the limited list, and the whole night is lost, even though it may be clear during a greater portion of the time.

At Stations No. 5 to No. 12, and at Station No. 15, the available stars formed a double (and in some parts even a triple) list of pairs extending through most or all of the hours of darkness. At those stations there was not the slightest difficulty in utilizing partially cloudy nights, or in utilizing all the hours of darkness on clear nights when there was necessity for rapid work. At Station No. 6, 49 observations were taken on June 14th, 1892, and 48 on June 16th, thus nearly finishing the work of the station in two nights. At Station No. 15, the observations were delayed greatly by clouds. When the weather at last cleared, 72 observations were taken on the nights of September 30th and October 1st, 1893, and the work at the station finished on the 2nd by 22 observations.

Taking most or all of the observations on one, two, or three nights, instead of extending them over from four to seven nights, will decrease the accuracy of the result if there are *constant errors peculiar to each night*—but not otherwise. The results as classified by nights, and given below, serve to show that for this series of observations the discrepancies between the results for different nights are no greater than should be expected from the *accidental* errors of observation and declination.

The column headed "Residual" shows the difference between the result for the night and the mean result for the station. The column headed "e_0 Obs. only" shows the probable error in the result for the night arising from error of observation only,—neglecting the declination error. This value is the probable error of a single observation divided by the square root of the number of observations on the night in question.

Station.	Date.	No. Obs.	Seconds of Latitude.	Residual.	e_0 Obs. only.
No. 1	Feb. 15, 1892	15	59.40	0″.00	±0.10
"	" 16, "	19	59.39	−0.01	0.09
"	" 18, "	15	59.37	−0.03	0.10
"	" 27, "	18	59.44	+0.04	0.09
No. 2	Mar. 21, 1892	17	00.38	+0.17	0.09
"	" 22, "	21	00.11	−0.10	0.08
"	" 25, "	1	59.64	−0.57	0.35
No. 3	Apr. 8, 1892	29	57.98	−0.07	0.06
"	" 9, "	9	58.28	+0.28	0.12
"	" 12, "	6	57.97	−0.08	0.15
"	" 13, "	2	57.78	−0.27	0.25
No. 4	Apr. 18, 1892	18	58.66	+0.11	0.09
"	" 20, "	18	58.29	−0.26	0.09
"	" 21, "	18	58.55	0.00	0.09
"	" 22, "	19	58.58	+0.03	0.08
"	" 23, "	19	58.52	−0.03	0.08
"	" 25, "	19	58.62	+0.07	0.08
"	" 26, "	19	58.69	+0.14	0.08
No. 5	May 26, 1892	29	01.96	−0.04	0.04
"	" 27, "	42	02.02	+0.02	0.03
"	" 28, "	1	01.77	−0.23	0.22
"	" 29, "	17	02.06	+0.06	0.05
"	" 30, "	6	01.85	−0.15	0.09
"	" 31, "	4	02.02	+0.02	0.11
No. 6	June 14, 1892	49	57.94	0.00	0.03
"	" 16, "	48	57.95	+0.01	0.03
"	" 21, "	5	58.13	+0.19	0.09
No. 7	July 7, 1892	1	56.68	−0.05	0.26
"	" 8, "	29	56.68	−0.05	0.05
"	" 9, "	8	56.77	+0.04	0.09
"	" 21, "	5	56.58	−0.15	0.12
"	" 26, "	21	56.86	+0.13	0.06
"	" 27, "	26	56.81	+0.08	0.05
"	" 28, "	9	56.62	−0.11	0.09
No. 8	Aug. 9, 1892	17	59.05	+0.04	0.05
"	" 15, "	37	58.93	−0.08	0.03
"	" 16, "	39	59.05	+0.04	0.03
"	" 18, "	7	59.11	+0.10	0.08
No. 9	Sept. 7, 1892	19	06.14	+0.07	0.04
"	" 10, "	22	05.97	−0.10	0.04
"	" 12, "	26	06.12	+0.05	0.04
"	" 13, "	34	06.08	+0.01	0.03
No. 10	Oct. 1, 1892	18	34.96	+0.12	0.05
"	" 4, "	21	34.73	−0.11	0.05
"	" 5, "	18	34.71	−0.13	0.05
"	" 10, "	18	34.84	0.00	±0.05

Station.	Date.	No. Obs.	Seconds of Latitude.	Residual.	e_0 Obs. only.
No. 10	Oct. 12, 1892	16	34.74	—0".10	±0."05
"	" 13, "	12	34.84	0.00	0.06
"	" 14, "	8	34.43	—0.41	0.12
No. 11	Nov. 2, 1892	27	57.49	+0.11	0.04
"	" 3, "	40	57.34	—0.04	0.04
"	" 5, "	32	57.32	—0.06	0.04
"	" 8, "	27	57.32	—0.06	0.04
No. 12	Nov. 19, 1892	26	04.65	—0.12	0.06
"	" 21, "	3	04.73	—0.04	0.18
"	" 22, "	35	04.81	+0.04	0.05
"	" 23, "	32	04.92	+0.15	0.05
"	" 26, "	25	04.67	—0.10	0.06
No. 13	Jan. 17, 1893	27	34.64	—0.05	0.05
"	" 18, "	28	34.70	+0.01	0.05
"	" 24, "	28	34.66	—0.03	0.05
"	" 25, "	22	34.81	+0.12	0.06
No. 14	Feb. 15, 1893	19	01.14	+0.02	0.07
"	" 16, "	6	01.08	—0.04	0.14
No. 15	Sept. 27, 1893	2	01.29	—0.07	0.16
"	" 30, "	37	01.38	+0.02	0.04
"	Oct. 1, "	35	01.42	+0.06	0.04
"	" 2, "	22	01.19	—0.17	±0.05

The mean of the "Residuals," or differences between the result for the night and the mean result for the station is 0".09. On 31 nights out of a total of 68 the residual was not greater than the probable error in the result arising from observation only.

Inasmuch as zenith telescope latitudes are nearly independent of refraction is it likely that there is any sensible error peculiar to the night in any case in which the conditions at the observatory are properly attended to,—accurate adjustment of instrument, level corrections kept small, as little difference of temperature as possible inside and outside of the observatory, etc.?

All the azimuth observations were made with Fauth repeating theodolite No. 725. The horizontal circle 25^{cm} (10^{in}) in diameter, is graduated to 5′ spaces and is read by two opposite verniers to 5″. It is furnished on the horizontal motions with axis clamps and tangent screws working against spiral springs. The telescope has a focal length of 41^{cm} and objective 45^{mm} in diameter. The eye-piece used magnified about 30 diameters, and is furnished with a micrometer with which the azimuth observations were made. The standard or yoke of the instrument is of aluminum.

The horizontal circle is covered by a protecting plate which served its purpose so well that during two years continuous service in a very dusty country, a part of the time on line or tangent work, so little dust and sand found its way to the graduated surfaces and the centers or spindles of the instrument, that it was never found necessary to take the instrument apart for cleaning, and the graduation was brushed but once,—a discoloration having gathered on a few lines only.

A curious feature of this instrument was its action when used as a repeater in measuring horizontal angles. In such cases there was always an apparent yielding of clamps and other parts which caused every measured angle to be *ten or twenty seconds* too large, when all final pointings were made with the tangent screws acting against their opposing spiral springs. The yielding seemed an elastic motion, not jerky, and was so nearly constant for any series of observations that it was eliminated within the errors of observation by the practice of "closing the horizon."

By "closing the horizon" is meant in this connection measuring *both* the required angle and its explement or difference from 360°,—always turning the upper motion over the measured angle in the direction in which the circle readings increase, and making all pointings with the tangent screws working *against* their opposing spiral springs. Such a procedure gives two values for the required angle, the directly measured value and the difference between the measured explement and 360°. The mean of these two is the true value of the angle free from the effect of the *constant* yielding of clamps and allied parts.

That this method of observing sensibly eliminated the remarkably large clamp error, was shown by the instrument giving consistent results in successive measurements of the same angle, by the satisfactory closing of triangles in which angles were measured with this instrument, and by the fact that in cases in which the *same angle* was measured with this instrument and with a *direction* theodolite the results agreed within the errors of observation.

The theodolite when in use for azimuth work was usually mounted on a wooden pier in the observatory tent. This pier was similar to that used for the latitude instrument, but smaller and lighter, and it showed as satisfactory stability as did that pier.

The mark used for azimuth work was an ordinary bulls-eye lantern shining through a hole one inch in diameter in the front of the small box which served to protect it from the wind. This light was placed at each station from one to three miles from the theodolite.

The time was obtained with sufficient accuracy for the azimuth work

by sextant observations of the sun's altitude. As all the azimuth observations were taken near elongation, errors in time affect the computed azimuth to a very slight degree.

The observations were taken with the eyepiece micrometer by the method described in Bulletin No. 21, December 12, 1890, of the Coast and Geodetic Survey.

On the first night after the pier and tent were ready a single pointing was made upon the star and the time and reading of the horizontal circle noted. The instrument was then left standing,—covered to protect it from dust and sand,—with the lower motion clamped and the upper one loose. During the next day the upper motion was set to such a reading computed approximately from the observation of the night before as would place the telescope in the vertical plane of the star about 30^m before or after the elongations at which the observations were to be taken. The azimuth light was then placed in this vertical plane at a distance of one to three miles from the station, according to the topography along the line of sight. The position of the light as thus located was always found to be sufficiently accurate.

All observations were taken near elongation, usually within one hour, and Polaris was used at all stations. The azimuth light having been previously placed nearly in the vertical plane of the star, the observations consisted simply of the measurement with the eyepiece micrometer of the small horizontal angle between the mark and the star, the chronometer time of each star pointing being noted. For each set of observations, the axis of the telescope being made as nearly horizontal as possible and the horizontal circles being clamped so that the line of collimation of the telescope was nearly in the plane of the mark, the routine of work was as follows: five pointings with eyepiece micrometer were made upon the mark, the telescope directed to the star and the striding level placed in position, three pointings were then made upon the star and the time noted for each by the recorder, the level was read and reversed, two more pointings upon the star with noted times, level read again, axis of telescope reversed in wyes, striding level placed in position, three more pointings upon the star with noted times, level read and reversed, two pointings upon star with times noted, last level reading, and finally five pointings upon the mark. Three such sets usually required from thirty to fifty minutes.

The special conditions that led to the adoption of the above routine of observation are: that the effect of uniform motion of the instrument in azimuth shall be eliminated from the result by the method of observing; that the bubble of the level shall have time to settle without delaying

the observer for that especial purpose; and that the observations of a set shall be completed as quickly as possible.

At Station No. 10, in addition to the observations upon Polaris, δ Ursae Minoris and 51 Cephei were also observed,—a separate azimuth mark being placed under each star.

The eyepiece micrometer was always used in the position in which increased readings of the micrometer correspond to a movement of the line of sight toward the east, when the vertical circle is to the east, and toward the west when the vertical circle is to the west.

The computation of the horizontal angle between the mark and the mean position of the star during the set was made in the same way at every station and is shown by the examples of computation given below.

The computation of the mean azimuth of the star during the set was made by three different formulæ at different stations.

For Stations No. 2 to No. 4, No. 6 to No. 9, No. 11, No. 13 and No. 14, the azimuth of the star was computed as follows:

Let ϕ = the astronomical latitude of the station;

δ = the declination of the star;

A = the azimuth of the star counted from the north;

A_e = the azimuth of the star at *elongation* counted from the north;

t_e = the hour angle of the star at elongation;

t = the hour angle of the star at observation;

$T = t_e - t$, or what is the same thing, the chronometer time of elongation minus the chronometer time of observation.

Then the azimuth of the star counted from the north is at the time of any observation

$$A = A_e - \sin A_e \cos A_e \operatorname{cosec}^2 t_e \frac{2 \sin^2 \frac{1}{2}\tau}{\sin 1''} (1 + \cot t_e \sin \tau).$$

A_e and t_e are computed by the formulae $\sin A_e = \sec \phi \cos \delta$ and $\cos t_e = \tan \phi \cot \delta$.

The factor $\sin A_e \cos A_e \operatorname{cosec}^2 t_e$ is a constant for the night.

$$\frac{2 \sin^2 \frac{1}{2}\tau}{\sin 1''} (1 + \cot t_e \sin \tau)$$

varies rapidly with the hour angle t and must be computed for each separate pointing on the star. It varies quite slowly however for changes in the declination of the star and in the latitude of the station. This term was tabulated for an arbitrary latitude and declination of Polaris for every 10^s change in the value of τ from -61^m to $+61^m$. It was found that the values of this table were so nearly exact for the *actual* latitudes of stations and *actual* declinations of Polaris at the time of observation that it could be used without sensible error at all the

stations cited above, and was so used. The mean of the ten tabular values corresponding to the ten pointings on the star during a set being called M, the mean azimuth of the star for the set becomes

$$A = A_e - \sin A_e \cos A_e \operatorname{cosec}^2 t_e M.$$

EXAMPLE OF RECORD AND COMPUTATION.

Station No. 14. February 11th, 1893.

Observations for azimuth of mark on Polaris near western elongation.
Chronometer error $= + 2^h\ 29^m\ 47^s.2$. One division of level $= 3''.68$.
One turn of micrometer $= 123''.73$. $\phi = 32°\ 29'\ 01''.12$.

Cir. E. or W.	Level Readings W.	Level Readings E.	Chronometer time.	τ	Tabulated term.	Micrometer Readings. On Star.	Micrometer Readings. On Mark.	
E	$7^d.2$	$6^d.3$	$8^h\ 47^m\ 48^s.5$	$57^m 34^s.5$	6496.9	$19^t.219$	$18^t.400$	Longitude 2^h 31^m west of Washington.
	7.2	6.4	48 31.0	56 52.0	6338.5	.189	.391	
			49 01.5	56 21.5	6226.3	.170	.387	
	+14.4	—12.7	49 47.0	55 36.0	6060.3	.146	.399	
E			50 20.0	55 03.0	5941.2	.124	.393	
						19.1696	18.3940	Means.
W	6.0	8.0	8 52 09.0	53 14.0	5556.9	17.790	18.470	
	6.1	7.8	52 36.5	52 46.5	5461.9	.800	.470	
			53 04.5	52 18.5	5366.1	.820	.469	
	+12.1	—15.8	53 57.5	51 25.5	5187.0	.852	.475	
W	—2.0	=Sum	54 28.0	50 55.0	5084.8	.871	.462	
					5772.0	17.8266	18.4692	Means.

α of Polaris $= 1^h\ 18^m\ 48^s.0$.

δ of Polaris $= 88°\ 44'\ 33''.4$.

$$\begin{aligned} \log \tan \phi &= 9.8039137 \\ \log \cot \delta &= \underline{8.3414167} \\ \log \cos t_e &= 8.1453304 \\ t_e &= 89°\ 11'\ 57''.5 \\ &= 5^h\ 56^m\ 47^s.8 \end{aligned}$$

$$\begin{aligned} \log \sec \phi &= 0.0738919 \\ \log \cos \delta &= \underline{8.3413121} \\ \log \sin A_e &= 8.4152040 \\ A_e &= 1°\ 29'\ 26''.34 \end{aligned}$$

$$\begin{aligned} \log \sin A_e &= 8.41520 \\ \log \cos A_e &= 9.99985 \end{aligned}$$

Constant for eight readings of level $= \dfrac{3.68}{8} \tan \phi = 0''.293$

$$\begin{aligned} \log \operatorname{cosec}^2 t_e &= \underline{0.00008} \\ &\ 8.41513 \\ \log 5772.0 &= \underline{3.76133} \\ \log 150.13 &= 2.17646 \end{aligned}$$

Level correction $=(-2.0)\ (0.293)= -0''.59$

Zenith distance of star = 57° 09′ (by computation.)

	1^h	18^m	$48^s.0$	$= \alpha$
+	5	56	47.8	$= t_e$
+	2	29	47.2	= Chronometer error.
	9	45	23.0	= Chronometer time of elongation.

Collimation reads, ½ ($18^t.3940 + 18^t.4692$)	$=18^t.4316$	
Mark W. of collimation, $18^t.4316 - 18^t.3940$	=0.0376=	04″.65
Circle E., star E. of collimation, $19^t.1696 - 18^t.4316$	=0.7380	
Circle W., star E. of collimation, $18^t.4316 - 17^t.8266$	=0.6050	
Mean, star E. of collimation	=0.6715=	1′ 38″.90
Level correction	=	− 0″.59
Mark west of Star	=	1′ 42″.96
Reduction to western elongation	=	2′ 30″.13
Marks east of western elongation	=	47″.17
A_e	=	1° 29′ 26″.34
Mark west of north	=	1° 28′ 39″.17
Mark west of south = Z′	=	178° 31′ 20″.83

In reducing the measured angle between the star and the line of collimation it must be remembered that the thread of the micrometer when not at the collimation reading describes a *small circle* on the celestial sphere as the telescope rotates about its horizontal axis.

Thus, star east of collimation, $= 0^t.6715 = (0.6715)\ (1.2373)$ *cosec* (zenith distance of star) $= (0.6715)\ (1.2373)$ *cosec* 57° 09′ $= 1'\ 38''.90$

In reducing the measured angle between the *mark* and the line of collimation cosec (zenith distance of mark) may usually, as here, be assumed unity, without sensible error,—the elevation angle of the mark being generally small and the line of collimation being purposely placed very near the mark.

The level correction for the *mark* is usually negligible.

At Station No. 1 the azimuth of the star was computed by the less accurate but simpler formulae

$$A = A_e - \frac{2\ sin^2\ \frac{1}{2}\ \tau}{sin\ 1''}\ tan\ A_e\,.$$

At this station no observation was made more than 26^m from elongation, and this formula was sufficiently accurate.

At Stations No. 5, No. 10, No. 12 and No. 15, the azimuth of the star was computed by the following formulae.

If A be the azimuth of the star, counted from the north, *corresponding to the mean hour angle of the set*, t,

$$\tan A = \frac{\sin t}{\cos \phi \tan \delta - \sin \phi \cos t}.$$

If A' be the mean azimuth of the star for the set, counted from the north,

$$A' = A - \tan A \frac{1}{n} \Sigma \frac{2 \sin^2 \frac{1}{2} \tau}{\sin 1''}$$

in which n is the number of pointings upon the star during the set, τ is here the difference between the time of any one observation and the mean of the times, and

$$\Sigma \frac{2 \sin^2 \frac{1}{2} \tau}{\sin 1''}$$

is the sum of the individual values of that factor corresponding to the individual pointings. The quantity,

$$\frac{2 \sin^2 \frac{1}{2} \tau}{\sin 1''}$$

may be found tabulated in convenient form in Doolittle's Practical Astronomy.

For this and other formulae for reducing azimuth observations see Coast and Geodetic Survey Report for 1880, Appendix No. 14.

EXAMPLE OF RECORD AND COMPUTATION.

Station No. 10. October 13th, 1892.

Observations for azimuth of mark on Polaris near eastern elongation.

Chronometer error = $+ 2^h 11^m 28^s.2$ One division of level = $3''.68$

One turn of micrometer = $123''.73$ $\varphi = 31° 19' 35''$

Cir. E. or W.	Level Readings W.	Level Readings E.	Chronometer time.	τ	$\frac{2 \sin^2 \frac{1}{2} \tau}{\sin 1''}$	Micrometer Readings. On Star.	Micrometer Readings. On Mark.	
E	$8^d.0$	$9^d.9$	$9^h\ 06^m\ 38^s.0$	$3^m\ 58^s.6$	31.05	$18^t.379$	$18^t.310$	Longitude 2^h 12^m west of Washington.
	10.0	7.3	07 32.0	3 04.6	18.59	.388	.315	
			08 05.5	2 31.1	12.45	.400	.315	
	+18.0	−17.2	09 13.0	1 23.6	3.82	.424	.311	
E			09 48.0	0 48.6	1.29	.430	.316	
						18.4042	18.3134	Means.
W	9.0	9.0	9 12 01.8	1 25.2	3.96	18.100	18.290	
	7.0	10.9	12 24.7	1 48.1	6.37	.100	.275	
			12 48.3	2 11.7	9.46	.090	.279	
	+16.0	−19.9	13 36.3	2 59.7	17.61	.086	.281	
W			13 58.1	3 21.5	22.14	.080	.279	
			9 10 36.6		12.67	18.0912	18.2808	Means.

Sum $-3^{d}.1$

α of Polaris $= 1^{h}\ 20^{m}\ 07^{s}.4$

δ of Polaris $= 88^{\circ}\ 44'\ 10''.4$

Constant for eight readings of level $= \frac{1}{8}\ (3''.68)\ \tan\varphi = 0''.280.$

Zenith distance of star at observation (computed) $= 58^{\circ}\ 47'$

$\log \frac{1}{n} \sum \frac{2 \sin^2 \frac{1}{2}\tau}{\sin 1''} = \log 12.67$	$= 1.10278$
$\log \tan A$	$= 8.40969$
$\log \delta A$	$= 9.51247$
δA	$= 0''.33$

Collimation reads, $\frac{1}{2}$ (18^{t}.3134 + 18^{t}.2808)	=18^{t}.2971	
Mark E. of collimation, 18^{t}.3134 − 18^{t}.2971	= 0.0163 =	02″.02
Circle E., star E. of collimation 18^{t}.4042−18^{t}.2971	= 0.1071	
Circle W., star E. of collimation 18^{t}.2971−18^{t}.0912	= 0.2059	
Mean, star east of collimation	= 0.1565 =	22.64
Level correction = (− 3.1) (0.280)		= − 0.87
Mark west of star		= 19.75

Mean chronometer time of observation	$= 21^{h}\ 10^{m}\ 36^{s}.6$
Chronometer correction	= 2 11 28.2
Mean sidereal time of observation	= 18 59 08.4
α of Polaris	= 1 20 07.4
t = 95° 14′ 45″.0 =	6 20 59.0

$\log \cos$ = 9.9315695	$\log \sin \phi$ = 9.71593	$\log \sin t$ = 9.9981771
$\log \tan \delta$ = 1.6563815	$\log \cos t$ = 8.96108 n	$\log d$ = 1.5884838
$\begin{cases} 1.5879510 \\ 38.72140 \end{cases}$	$\begin{cases} 8.67701\, n \\ -0.04753 \end{cases}$	$\log \tan A$ = 8.4096933
		A = 1° 28′ 16.″92
		δA = −0.33

Star east of north	$= A' = 1^{\circ}\ 28'\ 16''.59$
Mark west of star (from above)	= 19.75
Mark west of south	$= Z' = 181^{\circ}\ 27'\ 56''.84$

The same remarks as before apply to the reduction of the angle between star and collimation, and mark and collimation.

To all azimuths as above computed there must still be applied the small correction 0″.31 for diurnal aberration. For convenience this small correction was applied only to the final mean value.

The following table will serve to show the degree of accuracy obtained in the observations. e is the probable error of a single observation and e_0 is the probable error of the mean result for the station.

Sta.	No. of Nights of Observation.	No. of Observations.	Total Range of Results.	Range of Nightly Means.	e	e_o	
No. 1	2	5	1″.77	1.18	±0.46	±0.21	
No. 2	1	5	1.97	——	0.56	0.25	
No. 3	1	5	1.87	——	0.50	0.23	
No. 4	2	6	3.61	0.21	0.83	0.34	
No. 5	2	7	6.0	0.6	1.6	0.7	— Theodolite used as *repeater*.
No. 6	2	13	4.05	1.75	0.83	0.23	
No. 7	3	12	2.29	1.34	0.50	0.14	
No. 8	3	10	2.86	1.09	0.60	0.19	
No. 9	3	12	2.78	1.35	0.56	0.16	
No. 10	3	9	1.56	1.43	0.48	0.14	On Polaris.
No. 10	3	9	3.19	1.03	0.65	0.22	On δ Ursae Minoris.
No. 10	3	9	3.20	1.20	0.61	0.20	On 51 Cephei.
No. 11	3	9	2.69	1.17	0.60	0.20	
No. 12	3	9	3.53	1.76	0.77	0.26	
No. 13	3	9	2.00	0.34	0.42	0.14	
No. 14	3	9	3.00	0.90	0.64	0.21	
No. 15	3	9	3.92	2.28	±0.81	±0.27	

All the observations were made by the micrometric method except those at Station No. 5, which were made with the theodolite used as a repeater, just as in measuring any terrestrial horizontal angle.

The values of e and of e_0 as given above are computed by the ordinary formulae for independent observations of equal weight. In reality however several azimuth observations made on any one night at nearly the same time of night are likely to be affected by a considerable error common to them all,—a constant error peculiar to the night,—probably resulting from lateral refraction. Therefore several sets of observations taken in quick succession should not be considered to be independent. The values of e as given above are therefore slightly too small, and the values of e_0 much smaller than the reality. They serve however as a rough indication of the *relative* accuracy of the determinations at the different stations.

The probable error of observation and the error peculiar to the night of observation may be separated by the same method that is used in computations of zenith telescope latitudes for separating the errors of observation from the errors of declination.

Let e be the probable error of a single observation (i. e. one set), $\triangle$ be the difference between the value from each set and the mean for the night, $\triangle Z$ be the difference between the mean for each night and the mean for the station, and e_m the probable error of the mean result for the night.

Let e_n be the probable error peculiar to the night,—affecting equally all the results for that night.

Then e and e_n may be derived from observations at a series of stations treated as a single group, as follows:

$$e = \sqrt{\frac{0.455 \Sigma \Delta^2}{\text{No. sets} - \text{No. nights.}}}$$

$$\epsilon^2 = \frac{e^2}{\text{No. Nights}} \Sigma \frac{1}{n}$$

in which n is the number of sets on each night.

$$e_m = \sqrt{\frac{0.455 \Sigma \Delta Z^2}{\text{No. nights} - \text{No. stations.}}}$$

$$e_n = \sqrt{e^2_m - \epsilon^2}$$

All the observations for azimuth made with the micrometer (all stations except No. 5) when treated as a single group in this way gave for the probable error of observation of a single set $e = \pm 0''.54$ and for the probable error peculiar to each night $e_n = \pm 0''.38$.

It is probable that e_n would be larger for most regions than it is for this series of stations,—the conditions here being peculiarly favorable. The meteorological conditions were subject to few sudden changes, the air was very dry, and the line of sight to the mark passed over surfaces free of water and usually bare of trees.

These two values are important as showing how little is to be gained by increasing the number of sets of observation on a night. As fixed by these, the probable error of the mean result for a night is $\pm 0''.66$ when one set is taken, $\pm 0''.54$ for two, $\pm 0''.49$ for three, $\pm 0''47$ for four, $\pm 0''.45$ for five, and $\pm 0''.44$ for six.

The value of one turn of the eyepiece micrometer was first determined by measuring the same small horizontal angle (about 1°) with the micrometer and with the horizontal circle, using the theodolite as a repeater. Three later determinations were made by observing the transits of a close circumpolar star near culmination across the thread set successively at each half turn of the micrometer. The four values for one turn of micrometer as thus observed were 123″.91, 123″.80, 123″.44, and 123″.76, of which the mean is 123″.73.

The star being very nearly in the vertical plane of the mark at observation, and in fact usually being on *both* sides of that plane before the work of the night is over, the errors in the final result due to error in the assumed value of one turn of micrometer are very small in comparison with the errors of observation.

So, too, are the time errors very nearly eliminated by taking the observations quite near, and both before and after elongation.

At Stations No. 4 and No. 6, the mark was placed to the *southward* of the station, nearly in the vertical plane of the star, the topography being such that it could not be placed to the northward. Placing the light to the southward does not materially modify the method of observation or cause any special inconvenience.

The errors in the final mean for a station probably arise almost entirely from four definite sources. The errors of determining the inclination of the horizontal axis, the errors of the pointings, the instability of the instrument, and the error peculiar to the time of observation.

The first three are strictly instrumental errors and would be considerably reduced by using a larger instrument with a more powerful telescope. In forming an opinion of the merits of the micrometric method it should be remembered that the above azimuth observations were made with a small instrument (10-in. theodolite).

The fourth class of errors mentioned is due to causes which are external to the instrument and cannot be reduced by increasing the accuracy of the observations. The fact that such external errors exist and are not small in comparison with the instrumental errors, as put in evidence above, indicates that it is not so desirable to increase the accuracy of the observing as to increase the number of nights of work, and that a small instrument very convenient for transportation may give results of about the same degree of accuracy as a much larger and more unwieldy instrument.

For this series of stations a small instrument which was easily transported (even on pack mules in some cases), reduced the error of observation for a night by three micrometric sets of observations requiring from thirty to fifty minutes, to about the same magnitude as the external error known to be inherent in the night's work and which could not have been reduced by a better instrument and better observing.

* * * * * * * * * *

A portion of the international boundary three hundred and seventy-three kilometres long was located by the use of the same eyepiece micrometer that was employed for the azimuth determinations as described above. Heliotropes were used as pointing signals almost entirely, and also furnished a ready means of communication between the observer and the heliotropers.

The line was established in sections, each of as great length as possible, on which all points were established by running *toward* a fixed point. The front heliotroper, having placed his heliotrope over a distant point on the line, and the instrument being at a known point on the line, a second heliotroper placed himself approximately on range

between the instrument and the front heliotrope near the proposed site of a station and showed a light to the instrument. The small angle between the two heliotropes was then measured with the eyepiece micrometer. The observer next telegraphed the result to the near heliotroper, using the Morse alphabet by long and short flashes of a heliotrope kept at the instrument for that purpose. The message as received by the heliotroper indicated to him how many divisions of micrometer he was from the line and in what direction. Knowing that each division was equivalent to 6^{mm} per kilometre of distance between himself and the instrument, he converted divisions of micrometer to linear measurement, using his best judgment as to his distance from the instrument. This measurement he made upon the ground with a 20^{m} steel tape and placed his heliotrope at the new position. The process of measuring the angle between the two heliotropes and telegraphing the result was then repeated. The first offset as measured by the heliotroper necessarily corresponded to the difference on the micrometer indicated by the first and second messages from the observer, and therefore served to determine the distance from the heliotrope to the instrument. Using this known distance, the heliotroper computed the offset necessary to place him on the line and again placed his heliotrope by measurement. This process was repeated until the angle between the two heliotropes was apparently less than two divisions of micrometer. The final measurement of the angle by twenty-seven pointings with eyepiece micrometer on the two heliotropes ended the observations for that station. The pointings were taken in sets of three alternately on each light. As soon as the measurement was completed "O. K." was sent to the near heliotroper and he proceeded to mark the station and move on to the next.

Whenever convenient, as determined by the topography, position of camp, means of transportation, etc., the instrument was moved forward to some one of the newly determined points and the process of lining in points ahead was continued. In some cases the points, about two kilometres apart on the average, were lined in ahead from a single station for as much as fourteen kilometres. In other cases the instrument was moved up at every station so that it was never more than about three kilometres from the point being set.

In a few cases the instrument was lined in between two known points, one ahead and one behind, but this plan was avoided as much as possible because it was much slower than the usual method.

In many cases the section of the line established by running *toward* a fixed point was comprised between two intervisible monuments, and

the foresight was placed on the forward monument. In cases in which no monument was visible ahead, the line was run in the usual way to a foresight at the most distant visible point on the approximate line. From that point the line was produced ahead, again to the most distant visible point, using a backsight,—the process of using the micrometer being substantially the same as when running by foresights only. The intermediate points on this new section of the line were then located by running *toward* the newly located foresight as usual.

After the heliotroper first showed his light near a proposed station the time required to place his heliotrope within two divisions (2″.5) of the line and to make the final measurement of the small outstanding angle by twenty-seven micrometer pointings was usually from thirty minutes to one hour. Much more time was spent in traveling to and from stations than was required for the actual instrumental work.

The final measurements of the small angle between the two heliotropes made it possible, by the use of the distances afterward determined by the topographical party, to compute the small offset from the station as placed to the true line. These small offsets were computed and furnished to the monument party.

The work was pushed forward at all times without reference to favorable or unfavorable conditions for accurate observation. It was considered that such a procedure would give all needful accuracy and that economic considerations would not justify any additional expenditure. Much of the observing was done under conditions that at first sight would seem likely to lead to large errors, and it was therefore more than usually desirable that the accuracy of the determinations should be put in evidence.

A study of the method of work and of the instrument leads one to the conclusion that the errors in the location of the various points on the ground arise almost entirely from two causes,—first, instability of instrument during observations, and, second, errors of pointing.

Inasmuch as the instrument was simply mounted on its own tripod, and was without protection from sun or wind, it was subject to irregular movements due to its exposure. The routine of observation was especially designed to cancel out the effect of such movements upon the result. An idea of the magnitude of the errors arising from this cause can be gained by noting the average difference between each set of observations and the mean of the sets, as stated later in this paper.

The value of one division of the striding level was 3″.68. The level was applied so often, and the inclination of the line of sight was usually so small that the errors from this source must be almost inappreciable.

By the phrase, "errors of pointing," as used above, must be understood, not only the errors made in attempting to place the observing thread upon the brightest point of the heliotrope light as seen, but also the errors arising from the assumption that the apparent position of said brightest point coincides with the actual center of the heliotrope mirror from which the light issues.

From May 8th to the end of the work (September 15th) a record was kept during observations of the appearance of the heliotropes, of the temperature, and of the apparent diameter of the heliotrope light as measured in divisions of the eye-piece micrometer.

The temperatures were taken with a Centigrade thermometer placed in the most extensive shade available in the vicinity of the instrument, which was usually the little shadow cast by the instrument box. There are but few cases in which the recorded temperature at observation is below 25° C. (77° F.). During June and July the greater number of the recorded temperature were between 35° C. (95° F.) and 43° C. (109° F.). In a single case, on June 24th, between one and two P. M., the temperature indicated by the thermometer hanging in the shade of the instrument box was 48° C. (118° F.).

The apparent diameter of the light of the heliotrope in divisions of micrometer (each $1''.24$) varied from five or six divisions, in a few rare cases either in the morning or under a clouded sky, through intermediate values to the other extreme when the apparent diameter was 104 divisions. This last case occurred between eleven and twelve on a very hot July day on a line of sight about 2^{km} long, which was within 3^{m} of the almost bare sand. The average apparent diameter of the heliotrope lights for the season was 25 divisions ($31''$).

The diameter of the light, as measured, includes only what might be called the *solid* part of the light, and excludes the rapidly changing rays and spots of light which sometimes formed a further irregular extension.

The principal mirror of the heliotrope "R" in this paper was round, with a diameter of 10^{cm}, and that of the heliotrope called "C" was round with a diameter of $7^{cm}.5$. If the mirror itself were seen with the telescope it would appear as an ellipse with the major axis equal to the diameter of the mirror. The measured angular diameter of the light as seen, taken in connection with the known distance to the heliotrope, shows that the mirror itself was visible only on very rare occasions, even when the conditions were favorable. Usually, the apparent width of the object upon which the pointings were made, was many times that of the heliotrope mirror. For example, in the case where the light appeared 104 divisions in diameter, it corresponded to a target $1^{m}.0$ in

diameter placed in the position of the heliotrope 1600^m from the instrument, although the diameter of the mirror was only $7^{cm}.5$. On May 15th, with the temperature in the shade, 39° C. (102° F.) a heliotrope mirror 10^{cm} in diameter, 43^{km} distant from the instrument, showed as a blaze of light 44 divisions (55″) in diameter, corresponding to a target in the position of the heliotrope $11^m.4$ in diameter.

The characteristic appearance of the heliotrope light as seen during the observations, over highly heated and nearly bare sand or rocks, was a bright blur of light, with outline but poorly defined. Each separate portion of the outline seemed to be vibrating violently. Usually there was a part of the blur considerably brighter than the remaining portions, and the brightness increased gradually from the outline inward toward this region. In the most frequent case this bright region was in the centre of the blur, and the extent of the vibration on all parts of the outline was about the same. It frequently occurred, however, that the outline was decidedly unsymmetrical with respect to the brightest region. In those cases the brighest region was always *to windward* of the center of the blur and the *leeward* part of the outline vibrated more violently than the windward side,—the whole appearance being much like that of flames blown sidewise by the wind.

In all cases the pointings were made upon the assumption that the actual position of the mirror was represented by *the brightest portion of the blur* and not necessarily by the centre of the blur.

As a rule the violence of the vibration of the light increased with increase of apparent diameter. The size of the light and the violence with which its outline vibrated was apparently greater for lines which were near the ground than for high lines, and less during a steady breeze than when the wind blew in gusts and whirls or when the air was still. Evidently there were various other unknown causes producing large and apparently accidental changes in the appearance of the light. This makes it necessary to base any reliable conclusions upon a considerable number of observations at various stations and on many different dates. Accordingly in the investigation as to apparent diameter of light and accidental errors of pointing of which the results are given below all available observations were used, extending over a period of about four months.

The pointings upon each heliotrope were made in sets of three pointings each, taken in quick succession. A convenient measure of the accidental errors of pointing is the mean of the differences between each of the pointings and the mean of the three. This mean difference may be called $\triangle 3$.

Variation of Apparent Diameter of Light and of △*3 with the Time of Day.*

Heliotrope R.					Heliotrope C.				
Hours.	Diameter in Div's.	No. Obs.	△3 in Div's.	No. Obs.	Hours.	Diameter in Div's.	No. Obs.	△3 in Div's.	No. Obs.
6– 7	12	10	0.78	11	6– 7	18	9	0.57	11
7– 8	18	19	0.90	21	7– 8	28	23	0.84	27
8– 9	19	21	0.73	24	8– 9	25	37	0.74	41
9–10	21	21	0.94	26	9–10	27	38	0.91	46
10–11	31	17	1.12	21	10–11	34	23	0.85	27
11–12	35	8	1.08	8	11–12	31	18	1.00	18
12– 1	42	1	1.40	1	12– 1	22	3	0.76	5
1– 2	24	4	1.05	4	1– 2	49	4	1.05	4
2– 3	32	7	1.02	9	2– 3	28	16	0.88	16
3– 4	27	8	0.68	12	3– 4	23	12	0.69	14
4– 5	21	5	0.58	8	4– 5	19	12	0.68	14
5– 6	16	5	0.60	5	5– 6	20	5	0.47	6

The mean diameter of the light for Heliotrope R. was twenty-three divisions (29″) and for Heliotrope C. was 27 divisions (34″). The mean value of △3 was $0^d.88$ (1″09) for R and $0^d.80$ (0″.99) for C.

Very few of the observations were made within an hour of sunrise or of sunset. The table shows incidentally the hours most frequently used for observing.

Variation of apparent diameter of light for different distances between the theodolite and heliotrope.

Heliotrope R.			Heliotrope C.		
Distance. Kilometers.	Diameter. Divisions.	No. Obs.	Distance. Kilometers.	Diameter. Divisions.	No. Obs.
2	19	3	1	41	7
3	23	1	2	37	44
4	13	2	3	31	22
5	24	2	4	24	15
6	19	3	5	23	21
7	27	5	6	26	16
8– 9	26	4	7	17	12
10	20	2	8	27	18
11–14	26	18	9	16	4
15–19	23	30	10	24	6
20–24	26	19	11–14	24	10
25–29	11	12	15–19	18	14
30–34	17	7	20–24	19	4
35–39	35	7	25–29	13	7
43	33	5	1– 2	37	51
56	13	6	3– 5	26	58
2–14	24	40	6– 9	24	50
15–24	24	49	10–29	20	41
25–56	20	37			

Variation of $\Delta 3$ *for different diameters of light.*

Heliotrope R.			Heliotrope C.		
Diameter. Divisions.	$\Delta 3$ Divisions.	No. Obs.	Diameter. Divisions.	$\Delta 3$ Divisions.	No. Obs.
0– 5	0.77	4	0– 5	0.37	4
6–10	0.58	18	6– 8	0.45	6
11–15	0.70	20	9– 11	0.62	6
16–18	0.85	15	12– 14	0.65	18
19–20	0.95	4	15– 17	0.64	19
21–22	0.84	8	18– 20	0.68	24
23–24	0.66	5	21– 22	0.90	10
25–26	0.88	6	23– 25	0.67	26
27–28	0.92	8	26– 28	0.69	20
29–30	1.15	11	29– 30	0.70	5
31–32	0.83	7	31– 32	0.77	6
33–34	0.90	2	33– 34	0.63	4
35	0.83	3	35– 37	0.97	7
36–38	1.00	2	38– 40	0.65	4
39–44	1.40	4	41– 44	1.20	9
45–49	1.12	4	45– 49	1.23	7
50–60	1.90	2	50– 60	1.54	11
61–67	1.70	3	61–104	1.29	10
0–15	0.66	42	0– 20	0.63	77
16–28	0.85	46	21– 34	0.71	71
29–67	1.15	38	35–104	1.22	48

The table shows that the accidental errors of pointing increase much more slowly than the apparent diameter of the light in spite of the fact that with increased diameters there is also increased vibration, and shows that in so far as *accidental* errors were concerned it was safe to observe, even under the apparently abnormal conditions, when the diameter of the light as seen was from 1′ to 2′.

$\Delta 3$ equal to 1.2 divisions corresponds to $\pm 2''.7$ for the probable error of a single pointing.

The final determination of each station lined in ahead of the instrument consisted of three sets of nine pointings each. A convenient measure of the accidental errors of the sets is the mean of the differences between the result from each set and the mean of the three results. This mean difference may be called Δs. The actual relation between Δs and $\Delta 3$ is shown in the following table in which the first column is the mean $\Delta 3$ for the two heliotropes:

Δ_3 Divisions.	Δs Divisions.	No. Obs.	Δ_3 Divisions.	Δs Divisions.	No. Obs.
0.0–0.3	0.30	1	1.1	0.98	5
0.4	0.45	12	1.2	0.87	8
0.5	0.80	2	1.3	1.32	4
0.6	0.70	20	1.4	0.95	2
0.7	0.54	7	1.5–1.6	1.05	2
0.8	0.83	15	1.7–2.2	0.80	1
0.9	0.94	7	0.0–0.7	0.60	42
1.0	0.78	5	0.8–2.2	0.92	44

$\Delta s_{,} = 0^d.92$, corresponds to $\pm 2''.1$ for the probable error of a single set, and $\pm 1''.2$ for the probable error of the mean of three sets.

Δs is apparently proportional to $\Delta 3$, but is about three times as large would be accounted for by the influence of $\Delta 3$ alone.

Only a limited time was available for experimental work in the field, but the following evidence in regard to the magnitude of the *constant* or *systematic* errors arising from the use of heliotropes was secured as opportunities presented themselves.

On May 8th, in the Tule mountains, a series of observations were made for the purpose of determining whether there was any apparent displacement of the light when the heliotrope was neglected, and also to determine whether the apparent position of the light coincides with the actual position of the heliotrope mirror. The line of sight passed from one high ridge to another, about 4^{km} distant, over deep canons and intervening ridges reaching nearly up to the line of sight. Pointings were made in quick succession upon an one-inch pole carrying a flag and upon the heliotrope accurately in line with the pole and within two metres of it. The mirrors of the heliotrope having been adjusted, the observations continued for fifteen to twenty-five minutes before the mirrors were again readjusted. During the observations the heliotrope light appeared to be from four to fifteen divisions in diameter. The whole series of observations comprises 47 pointings on the pole and 195 upon the heliotrope, extending over a half hour in the forenoon and about two and a half hours in the afternoon. The mean result from all the observations is that the heliotrope light seemed to be 0.15 division $=0''.19=4^{mm}$ south of the pole. The observations do not show conclusively that there was any apparent change in the position of the light when the mirrors were neglected. If there was any such change it was masked by the accidental errors of observation. The accidental errors of pointing seemed to be slightly greater when the mirrors were first adjusted than later when the light had become less bright and apparently smaller.

On June 7th, at Station VI of line B, twenty sets of observations of twelve pointings each, extending over four hours, were taken to ascertain the effect of reducing the mirror on Heliotrope C. from $7^{cm}.5$ in diameter to $2^{cm}.5$. Heliotrope C. was at Station IV, 5800^{m} to the westward of the instrument, and Heliotrope R. was at Monument IX, 17400^{m} to the eastward. The average apparent diameter of the light from Heliotrope C. was 25 divisions when the full size of the mirror was used and 23 divisions when the light was reduced by passing through a hole $2^{cm}.5$ in diameter. The reduced light seemed to be but slightly less bright than the full light. The accidental errors were as great with the reduced as with the full light. The mean of the ten sets, with the full light, gave Station IV $2^{d}.8$ north of the line Monument IX to Station VI produced, and the ten sets with reduced light made it $3^{d}.1$ north. There was a remarkable difference between the forenoon observations and those made in the afternoon. The mean of all sets in the forenoon made Station IV $5^{d}.9$ north of the line Monument IX to Station VI produced, and the mean of all sets in the afternoon made it but $1^{d}.0$ north. The original determination of the position of Station VI from Station IV, on June 5th, would make Station IV $0^{d}.2$ north of the line Monument IX to Station VI produced. This extreme range of 5.7 divisions in the three values corresponds to a range of 20^{cm} in the position of Station IV.

This remarkably large range of results led to a special investigation at Monument IX, on June 15th, to determine whether there was a systematic difference between forenoon and afternoon observations, and whether there was any systematic error due to the position of the back glass, which was used to reflect the sunlight to the main mirror of the heliotrope when the sun was behind the heliotrope from the observer. Eight sets of nine pointings each were taken in the forenoon and fourteen sets in the afternoon. Heliotrope C. was used at Station IX, line B 7700^{m}, to the westward of the instrument, and its position compared with that of two flag poles nearly in line at Stations X and XI. The apparent diameter of the light during the observations was from 16 to 24 divisions. Taking the pointings upon the flag poles as a standard, the heliotrope appeared to be $0^{d}.6$ farther south in the forenoon than in the afternoon. During the forenoon observations the sun was to the northward of the line of sight and during the afternoon observations to the southward. In the afternoon every alternate set was taken with the back glass to the southward of the line of sight and the remaining sets with it to the northward. To exaggerate the effect, if any, the back mirror was purposely placed so that the line joining the centres of the two mirrors made a horizontal angle of about 50° with the line of

sight from the instrument. The mean of the seven sets with back mirror south agreed exactly with the mean of the seven sets with back mirror north.

For the purpose of detecting *constant* or *systematic* errors, the line from Station IV, line E, to Station XV 25800^m long, was located independently, both by lining in points ahead in the usual way and by lining in the instrument between two known points, one ahead and one behind. Errors of pointing upon the distant forward heliotrope will affect both these locations alike. But errors in pointing upon the other heliotrope will have contrary effects in the two cases, making the local line too far north in the first case and too far south in the second case if the heliotrope appears to be farther south than its true position. The greatest difference between the two locations at any of the ten intermediate stations was 32^{cm}, and the average difference 17^{cm}. With one exception all stations as located by the second method were farther south than by the first method. During the observations for these locations the heliotrope light appeared more than 30 divisions in diameter during a third of the time, and on one occasion appeared 104 divisions in diameter. The lines of sight were usually near the ground, there were several cases in which the light appeared decidedly unsymmetrical, and the conditions generally were as unfavorable to accuracy as on any part of the whole line.

As one more evidence of the degree of reliability of pointings upon heliotropes under such unfavorable conditions as those encountered during this season may be mentioned the azimuth error developed by the azimuth connection at the end of the line. The first section of the line had a known azimuth determined by astronomical observations and triangulation at and near Yuma, Arizona, and the azimuth of the last section was determined both by the distances and angles along the line as run and by astronomical observations and triangulation near Nogales, Arizona. The accumulated difference between these two determinations of the azimuth of the last section of the line arising from errors in the astronomical observations, errors in the two triangulations, and errors in the measurement of sixteen angles along the line, was 6″.7. The measurements of the sixteen angles along the line were made upon heliotropes under conditions no more favorable to accuracy than during the other portions of the work.

No attempt has here been made to account for the facts noted as to the appearance of the heliotrope light. What principle of optics will account for the great size of the light as seen in comparison with the mirror from which that light actually issues? How can the influence of the wind in making the light appear unsymmetrical be explained?

Some Notes on Fire Protection Engineering.

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Within recent years attention has been directed to a field of engineering that received but scant and irregular attention in the past, but which is coming to be actively worked and is each year yielding a richer harvest. It is a broad field, there are great financial interests at stake and the skilled workers in the field are few; it has not yet found proper recognition in the text-books. I therefore feel that in this lecture your attention may be profitably directed to what we may name "Fire Protection Engineering."

THE ANNUAL FIRE LOSS.

The value of the buildings and their contents destroyed by fire in these United States in the year 1891 was 144 million dollars,—in 1892, it was 152 millions and in 1893, nearly 160 millions. There are further losses resulting directly from the fire hazard: thus it costs each year about 40 million dollars to meet the commissions, salaries and office expenses of the insurance officials engaged in the distribution of this loss, and there is a still further tax of about 30 million dollars per year upon the resources and production of our country to meet the expenses of public fire departments.

Thus we see that the burden arising from the fire loss is great, and that it is well worth careful study to reduce it or even to hold it from becoming greater. The increasing height of city buildings under the stimulus of the perfected passenger elevator, and the greater and greater crowding and concentration of value within factory yards and along city streets, the higher speeds of machinery, the effort to make the processes of industrial chemistry more rapid, and the introduction of electric currents into buildings of every kind renders the problem more difficult year by year.

What can we do about it? How far will it pay to go in spending money to secure greater safety?

Economics of Fire Prevention.

Many of the economists who grieve over the great annual fire waste forget that to burn up a dollar may be no more unprofitable than to bury it beyond sight and circulation in an over-expensive incombustible structure.

There are many methods of construction of undoubted safety, which must be rejected because we have not sufficient money in hand to permit adopting them: it would have absorbed far more capital to have constructed our buildings like the European models which these economists so highly approve than the saving in fire loss and the saving in cost of fire protection and insurance would justify.

Take the case of a Cotton Mill. To build this with the fire-proof construction, the iron floor beams and the arched brick floors now common in modern English factories would increase the cost 40 cents per square foot of floor above that of a typical modern American factory with plank and timber floors and wooden pillars; but by spending 5 cents per square foot of floor for automatic sprinklers, fire-pump and private hydrant system, the American factory, judged by the insurance cost becomes just as safe against fire as the English fire-proof mill. We might cite other illustrations to prove that expensive European models were not the best for America with its cheap lumber and high cost labor and that care and intelligence in fire protection may often save unnecessary expense in construction.

It is an indisputable fact that this American fire loss of 150 million dollars a year, is largely an unnecessary loss. For factories and warehouses it has been amply demonstrated that a structure which is reasonably safe can be built at a cost which is not appreciably greater than the cost of the more common and unsafe structure, if the architect or engineer but knows how to go to work. The safety of dwellings from fire can perhaps be doubled by precautions so cheaply taken that their extra cost could hardly be found in the contractor's estimate. The President of a single large insurance company in New York City states that his one company pays losses amounting to two hundred thousand dollars per year on fires originating from defectives flues alone, and that forty per cent of all its losses are due to the defective condition of the building in which the fire happens to originate and which allows it to spread. There are many lines of business where a well planned installation of hydrants, automatic sprinklers and the like,—thereby increasing the cost of the whole plant from 2% to 4%, will not only soon pay for themselves in the lessened cost of insurance, but more important by far, will lessen the chance of interruption to business.

In studying how this great fire loss of one hundred and fifty million dollars per year is to be reduced, we must start with a clear understanding that the safety or degree of protection toward which we are striving, is comparative, not absolute. Safety so far as the value at risk is concerned may be had in two ways,—by preventing fire and by insurance. Whatever is burned is lost,—so much accumulated effort is wiped out,—absolutely wasted. The waste is just as great when fully insured as when not insured at all.

It may not be amiss to briefly define the ordinary function of an insurance company. The insurance company is merely a device for distributing a loss. Years ago when a farm house was burned, some kind friend "passed around the hat" among neighbors who thus each took a share of the loss,—now the Insurance Companies are elaborate organizations for the same purpose. The Mutual Company takes up its collection at the beginning of the year, pays out from it the amount of each loss as it occurs, and at the end of the year after taking out wages for its officials, divides up what is left and hands it back in just proportion to those who contributed it. The Stock Insurance Corporation puts up its own money in betting with you at odds of 20 to 1 on your carpenter shop, or gives you odds of 700 to 1 on your residence, that these will not burn during the next 12 months, and they hold the stakes. With the stock company just as with the mutual company, it is the thousand who have no fire who pay the loss for the one who suffers from fire. The underwriters estimate the chances on the basis of carefully compiled statistics and a vast experience, and yet the American fire waste is so great that in the long run they frequently lose instead of winning. Those companies writing the most risky structures, have commonly found notwithstanding the higher rates, that betting a poor risk would not burn, was in the end a losing operation.

At the close of 1891 with its 144 millions burned, 50 Insurance Companies went out of business. There has been for five years past a continual shrinkage of insurance capital and to-day many concerns with large warehouses in the conflagration districts of large cities find it absolutely impossible to obtain a proper amount of insurance, which they desire and are ready to pay for.

In studying fire prevention, the starting point must be to learn where to draw the line between preventing the possibility of a loss or on the other hand, prudently arranging to distribute the loss. The fruits of industry may be just as much wasted when apparatus is installed which will never be used, as when a stock of goods is burned. "Will it be worth its extra cost?" is a question which we must continually be asking regarding devices whose reliability and extra safety is beyond question, and

very often indeed will sound judgment decide that it is expedient to trust to luck and to the insurance policy rather than spend money in some construction or appliance of undoubted excellence which would probably not be put to the proof once in a score of years. Sometimes young concerns with scant capital may find it sound business policy to put their first money nearly all into productive plant and pay high premium for full insurance until the success of their new venture is assured.

On the other hand, to judge the value of a method of construction or of a fire appliance solely by comparing the annual saving of insurance premium due to adopting it, with the annual expenditure for interest on its extra cost, though often argued, may be a narrow and mistaken view, for there are losses which insurance does not cover and which the better fire protection may prevent.

To illustrate,—suppose we are building an isolated frame dwelling-house of ordinary construction and within the outskirts of a city like Ithaca, beyond the immediate protection of the Public Fire Department, which will be worth with its contents, $7000. This can be insured at an annual cost of not far from ten dollars per year. Or $200 placed at 5 % interest will yield an income sufficient to pay for the insurance. Therefore if an alternative mode of construction was presented which would render building and contents absolutely incombustible, we could not, to secure this fire-proof quality,—looking at it solely as a matter of dollars and cents,—afford to increase the cost of this dwelling more than $200 over that of the easily destructible house.

The above lesson holds good as a broad, impersonal average, such as appeals to the economist,—but the man driven from his house by fire at midnight, with loss of valuable papers and with the collections of a lifetime in personal treasures destroyed, will feel another truth and learn a lesson which will sink much deeper;—that the interruption and sorrow attendant on a fire may be tenfold greater loss than the value of the property burned.

A bad fire interrupts the economical routine of business, breaks contracts, scatters skilled workmen and drives customers to other sources of supply. At one of the largest paper mills, a fire which shut one department down for a few months sent a most profitable line of goods to other factories, for although this concern had previously been considered the only one that knew how to make this particular line of goods properly, somebody else *had to* find out how to make it when the first mill was out of the market. The customers got various mills to experimenting and the original concern never regained much of the trade. A prominent hardware manufacturer told me when I visited his new

factory a year after fire had destroyed the old,—"I find my line of customers is burned up as completely as was my factory, my competitors have captured my trade and I must make a sad break from the old prices, to get back." These are losses which insurance does not cover.

At a cotton mill making staple print cloths,—a wood-worker making doors,—a flour mill on ordinary grades,—or at any concern whose machines and product can easily be replaced, the uninsurable loss from interruption will not justify so great an expenditure as would be proper at a factory on special work, crowded with contracts or whose delicate special machinery it might take years to replace, or whose skilled workmen might find place with a competitor if laid off for a season.

The Reasonable Extent of Special Fire Protection.

To the question,—"How far will it pay to go, in incurring extra expense for fire resisting construction and for fire extinguishing apparatus?"—we will at this time answer only by a few illustrations and statements in which I believe that every competent engineer or underwriter who has seriously studied these problems will concur.

I. In Dwellings, Fire Stops.

In isolated frame dwellings, *it pays* when building to give greater attention to the chimney construction and to improve radically upon the ordinary fire stops in the partitions and floors, and to avoid studiously every unnecessary concealed space.

It is well proved that rats and mice have a strange fondness for the odor or flavor of common friction matches, or it is certain that these are often found in their nests. I myself, but a few days ago took more than a dozen friction matches from a mouse nest in a warm hollow near the heating pipe in an extremely valuable building. Such nests are commonly of very combustible material, and undoubtedly nests of mice in hollows of walls or floors, or in a warm corner against a chimney, are the cause of many fires which are recorded as of mysterious origin. Therefore every barrier to the passage of rats and mice,—every absence of a hollow space in which they can find a home, lessens the fire hazard.

Fire-stops receive far too little attention during dwelling house construction, and are passed over much too briefly in the ordinary architects' specification. They cost very little,—the stock to be used is a few waste brickbats, and a little coarse lime mortar. The only difficult thing to secure is the careful supervision which will make it absolutely cer-

tain before the lathing and plastering of walls and ceilings is completed, that the hollow spaces between the studding in all the walls and partitions is for about 8 inches in height close to each floor level filled conscienciously with brick and mortar so that there is not a crevice left open through which even an ant could crawl; and which will make it certain that the spaces between the ends of the floor joists where these join on to the walls and partitions, are all similarly filled for a width of about 8 inches

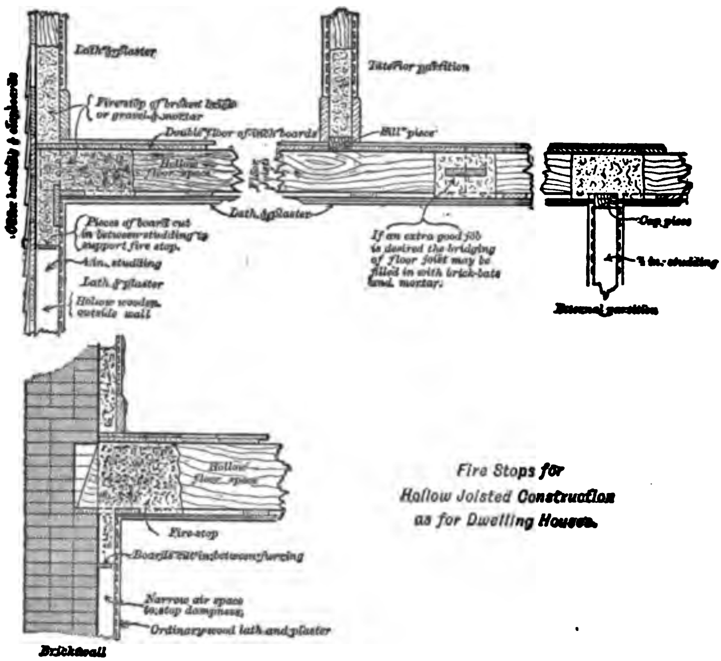


FIG. 1.

out from the inside of the partition. It would appear that thorough work on these fire-stops can be secured only by doing it all under the eyes of a special inspector and by a clause in the specifications requiring this work to pass under written approval before it is covered up and concealed from examination. These *fire-stops* are *vermin-stops* as well and also add to the warmth of the structure by preventing air currents.

Probably in not one house in twenty as built in the past or as built to-day, do the fire stops receive proper attention.

Beyond prudence in chimney construction and this continual vigilance regarding the construction of fire-stops, there is little in the fire protection of a dwelling which it will commonly pay to undertake; still

a few automatic sprinklers around the furnace and ash barrels in the cellar may not be amiss, and the writer believes that for the ground floor a plank and timber "mill floor" instead of the common joisted floor would in a fine house, be true economy.

II. In Factories and Warehouses, Slow Burning Construction.

For factories and warehouses *it pays* to adopt "slow burning" construction or "mill construction" as it is sometimes called, instead of building joisted floors and roofs covered with inch boards. This is illustrated in Fig. 2, next page.

The first characteristic of slow burning construction is that every hollow or concealed space is studiously avoided — the second is that the wooden skeleton which gives strength to the structure is massed into the fewest possible pieces, each of large size and exposing relatively small surface to the attack of the flames.

The floor beams are massive timbers about 12 inches wide by 16 inches deep, placed 8 or 10 feet apart, and the floor itself is of 3 inch or 4 inch spruce plank covered by layers of tarred paper to make it water tight, surmounted with a $\frac{7}{8}$ inch hardwood top floor, the whole making a solid layer 4 or 5 inches thick. Contrasted with this, the common hollow joisted floor has for its skeleton 3″ x 12″ beams placed 12″ or 16″ apart, and exposing from 4 to 6 times the amount of surface to the flames. So it happens that a joisted floor will collapse in 20 minutes over a fire which would not bring down or burn holes through a plank and timber floor of equal cost in two hours.

In the best mill construction the walls are of brick, bare on the inside, the roof is flat, and the cornice is preferably incombustible. The 3 inch plank roof has body enough to prevent warping and consequent injury to the impervious covering, and it is found that this thickness of wood is as good a non-conductor of the summer heat or the winter's cold as the more easily destructible hollow board roof with its air spaces.

Trussed roofs are shunned and openings in floors whether belt holes or for elevators or for inside open stairways, are all studiously avoided. It is found to be good economy to build outside stairway towers and thus avoid all openings in floors by arranging the ground plan as illustrated in Fig. 19, page 113.

This order of architecture is simplicity itself, but the evolution of the simple and massive from the more complex and flimsey required 40 years of disastrous fires, slow experience, and a continual study for safety, to develop and introduce it; and to-day it is but just beginning to be recognized outside the immediate neighborhood of the New England textile mills.

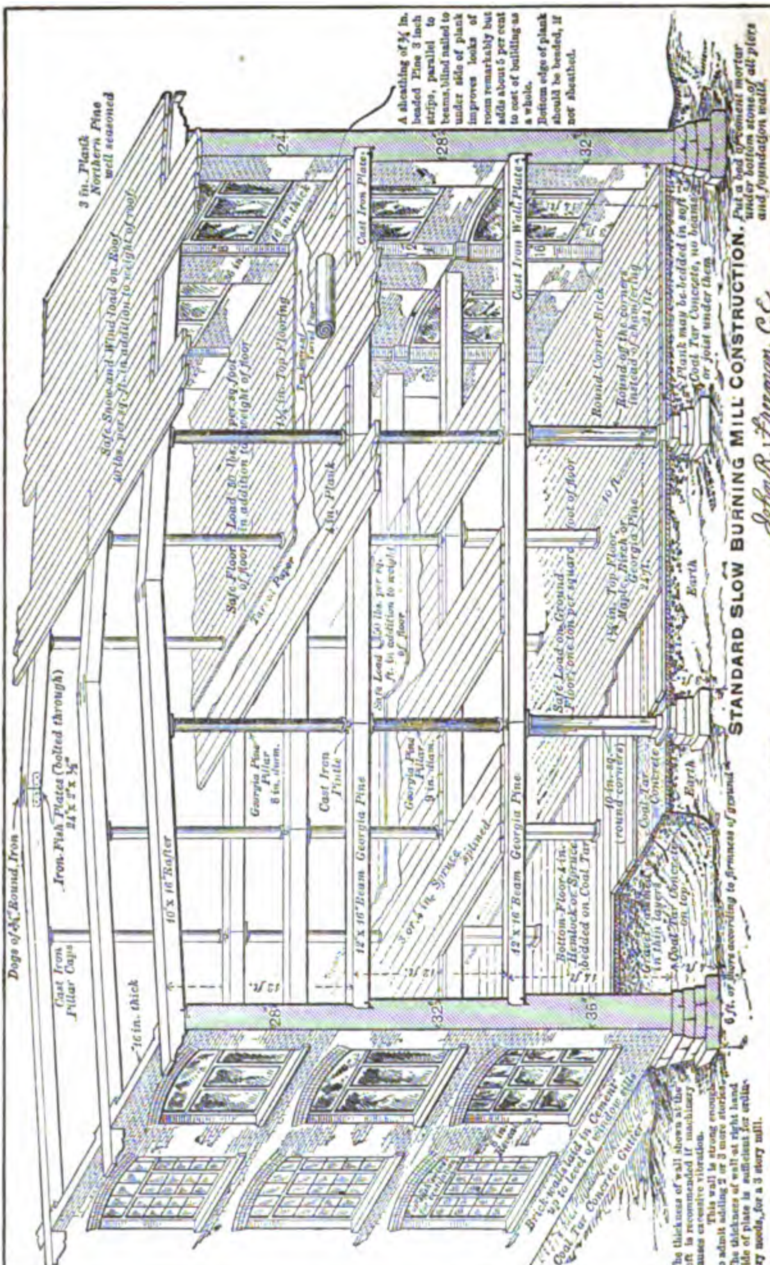

STANDARD SLOW BURNING MILL CONSTRUCTION.
John R. Freeman, C.E.
3 in. Plank Northern Pine well seasoned
Cast Iron Pillar Caps
Cast Iron Pintle
Cast Iron Plate
Cast Iron Wall Plate
Round Corner Brick
Earth

FIG. 2.

III. Fire-Sprinklers in Factories.

In factories of any considerable size or value where inflammable stock like wood, wool, cotton, hemp, cereals or sugar is worked, *it pays* to protect *every square foot of flooring* by automatic sprinklers.

An automatic sprinkler system consists of a pipe system with branch water pipes of wrought iron attached to the ceiling of the room and with sprinkler-heads placed 10 feet apart in both directions, all over the top of the room and fed by the pipe system just mentioned.

The sprinkler-head consists of a water discharging nozzle ½ inch in diameter, held closed by a cover until fire occurs and a link of easily fusible solder which melts at a temperature of 162° F. Various forms of sprinkler are shown on page 135.

The first successful automatic sprinkler was invented only 17 years ago and was first installed on a large scale by one of the leading manufacturers of Fall River, throughout the picking, carding and spinning rooms of three cotton factories then under his management. In doing this he was influenced by his own engineering judgment of the value of the device, without encouragement from the insurance Companies, and indeed for two years no rebate in insurance premium was allowed for this extra protection. Soon afterward the recognition of the merit of automatic sprinklers became common among the cotton manufacturers, and to-day there is hardly a cotton factory or a woolen factory of noteworthy size to be found in the United States or Canada which is without automatic sprinkler protection throughout the more hazardous half of its rooms. *In a large majority of best modern mills every room in which manufacturing is carried on is protected by automatic sprinklers of one or another of the kinds shown on page 135.*

In paper mills, notwithstanding that the stock is worked wet in many of the processes, it has been found true economy to install sprinklers in all rooms, even over the bearings of heavy shafting in wet and nearly vacant basements.

The men who have followed the records of fires most carefully among the factories insured in the Factory Mutuals, are convinced that it would be economy to mill owner and underwriter to forthwith install a sprinkler *over every ten-foot-square of factory property not yet so protected,* whether paper mill or textile mill, jewelry factory or machine shop, but as a matter of business policy, it is deemed best to treat the question in an advisory, rather than a mandatory way, and thus take several years to reach the end now clearly in view.

We may draw illustration from the records of a single insurance association: A loss of $300,000 occurred not long ago from fire origin-

ating in a wet turbine room containing also the fire pump, which room it had not been considered worth while to protect by sprinklers. The fire here gathered such force that when it passed into the Main Mill, the tank supplying the sprinklers was quickly exhausted and the whole factory was soon in ruins. There is no doubt in the minds of the ex-experts who examined the wreck, that a dozen sprinklers in this small turbine room would have held the fire in check aud saved the mill. The same year's record includes a $40,000 paper mill loss from fire originating in a basement where sprinklers were deferred because it was expected to remain vacant; a large dyehouse loss which there is every reason to think a few sprinklers would have saved; a disastrous cotton mill fire which all concerned think chargable to sprinkler protection having been too long deferred. So the list grows and counting only those cases where, judging by abundant parallels, sprinklers would have held the fire under control, *the record of a very few years shows a loss sufficient to equal the cost of extending sprinkler protection throughout all parts remaining unsprinkled in each of the many factories insured in the association whose experience we are quoting.*

IV. Sprinkler Protection in Commercial Buildings.

Sprinklers may prudently be installed in those commercial buildings of large floor area which are situated in the crowded centers of value in large cities.

It is only within 5 years or less that their value for this purpose has become widely recognized. The proprietors of large city stores filled with thousands of dollars worth of silks, furnishings, shoes, or a hundred things susceptible to heavy damage by water, naturally hesitate about placing a net work of water pipes over the ceiling with a sprinkler at every ten foot square, which, should it burst or be knocked open by the boy with the broom, might quickly cause a water damage of a thousand dollars. There are many departments in textile mills where, should a sprinkler let go prematurely, the loss would be equally severe, but experience in these mills has shown such accidental water damage to be so infrequent, that it is not now viewed with any noteworthy apprehension. There are many mills which have had a thousand sprinklers in place for 5 or 10 years past, where not a single case of premature discharge or damage from leakage is yet known, and in those instances where damage has occurred,it has commonly come from some new form of device or a too cheap installation.

There is probably no way in which the present conflagration hazard in the "dry goods district" of New York or Boston where values are concentrated sometimes to the extent of from one to two million dollars

per acre could be so economically reduced as by a general introduction of automatic sprinklers throughout those districts. The cost is only about 3 cents per square foot of floor and probably 3 to 5 years rebate in insurance cost would pay for completely sprinkling the average city square in which such extremely large values of goods are stored.

V. Private Fire Hydrants and Fire Pumps.

In nearly all large factories where the value of adjacent buildings and their contents is above $75,000 *it pays* to install a private system of fire hydrants and fire hose, and feed these by a source of water at as near 100 lbs. pressure as it is practicable to obtain and to have this independent from the public supply, or to provide two independent private sources of water under pressure for fire, as for instance by two separate pumps, if their is no public supply.

In large factories such as those joining in the Factory Mutual insurance system where the fire protection is looked after in about two thousand factories valued at seven hundred million dollars or on the average at about $300,000 each; *the experience from 10 to 40 years has demonstrated that complete automatic sprinkler protection is advisable; that two absolutely independent sources of water supply, each of about 1000 gallons per minute capacity at and upward of 75 lbs. pressure, should be insisted upon, and that a private system of hydrants should be so spaced and located that the whole volume of water for hydrant purposes can be concentrated on any one important building, with lines of hose averaging not more than 150 ft. long.* In matters of fire protection it has been common for a half-million-dollar factory to spend ten thousand dollars in fire protective apparatus which did not produce a yard more of cloth or one piece more of metal work.

The mill owners on whose judgment this expensive protection was installed are the chief captains of industry of New England, and one proof of their wisdom is found that in their Mutual Insurance Association *the mere saving in cost of insurance brought about through this fire protection, compared with the cost of the fire hazard without such protection, has commonly paid for the whole fire protection outlay in about three years,* and the danger of interruption to business by fire has at the same time been greatly reduced.

Development of Fire Resisting Construction in the U. S.

The fear of fire should be a leading motive in all architecture, but although the people of our nation waste ten million dollars a month as the mere value of the fuel on which our conflagrations feed, the extent to which inferior models are still followed in hundreds of industrial

works and public buildings which might be made safe at small increase of cost shows that there is still a sad lack of popular information on how to gain increased safety cheaply. The cause of this popular indifference to safer construction doubtless is that a serious fire comes so seldom to the average man that he under estimates the value of avoiding it or of building his structure in such shape that a slight fire cannot readily become a serious one.

The special study of fire prevention in America first took definite shape and became popular among the large New England cotton factories. The architect of 40, 20, or 5 years ago, took little interest in these innovations, and the building of large factories in New England became no longer entrusted to architects but to *mill engineers*, to men who were more concerned with a simple, strong and safe interior than with the appearance of the outside.

Lint cotton is a most inflammable substance,—it is worked in large masses on high speed machinery. A spark lodged in a bunch of lint cotton may burrow and smoulder unseen for days and then suddenly burst forth in flames. The undivided floor areas in cotton factories must for economy of manufacture be oftentimes tenfold greater than those at which the city underwriter shakes his head with foreboding.

About 50 years ago a certain progressive cotton manufacturer in Rhode Island,—Zachariah Allen,—built his new mill in the best and safest manner of which he could learn or devise, and asked for an insurance rate lower than the customary one. It is reported that he was answered, "A cotton mill is a cotton mill and the regular rate is $2\frac{1}{2}$%." He then associated himself with other manufacturers to form a Mutual Insurance Company which should recognize improvements in construction and in fire protection; this company prospered, others were formed, that same mill is still standing and for 20 years past the annual cost of insurance upon it has been less than one-tenth part of the old time standard rate of $2\frac{1}{2}$%. A few other Mutual Companies were organized and each was closely looked after by a Board of Directors everyone of whom was a practical manufacturer. These men were thus set to watching the fire hazard in their neighbors' mills even more than in their own,—a point of view affording a much greater advantage for just criticism,—and acting as insurers, were continually receiving object lessons in fire hazard and fire protection. At the regular monthly meeting of the practical mill men composing their boards of directors, the conversation was naturally directed towards improvements, and each came to take pride in efficiency in guarding against fire. Through their travelling inspectors, an improvement evolved in one mill was carried into notice at a great many others and attention was earnestly directed toward fire preven-

tion all along the line. Thus it came about after a score of years that the cotton mill with its dangerously inflammable contents became actually safer and a better insurance risk than the factory which was working incombustible stock like iron, or forming paper from watery pulp.

These Factory Mutual Insurance Associations carried on the business in only a small way for many years and in but a limited territory, but they were almost unconsciously all the time, directing attention to safer methods and to better protection, and they were unquestionably the pioneers in developing slow burning construction and a private fire protection of real efficiency. In later days these associations, instead of sitting down and waiting for fire to occur and then dividing the loss according to the old time insurance idea, have had for one of their chief functions the study of fire prevention and the demanding of the installation of such devices that a small fire should have almost no chance to become a big one.

The larger stock insurance companies have now begun working for better construction and more efficient protection with great earnestness. A system of schedule rating is just now being introduced in the larger cities, which, basing the insurance rate strictly upon the character of the construction, protection and occupancy, is certain to force a prompt recognition of the means of safety, against the indifference of builders and owners. The outlook for the general recognition of safer methods of fire resisting construction was never brighter than to-day.

Fire-Proof Structures.

Probably no commercial building yet devised would prove strictly fire-proof if crowded with blazing merchandise and on fire fiercely with a free air supply for two or three hours, or until the iron skeleton should become heated enough to expand and warp — but the term fire-proof building is nevertheless a proper one to use, and defines clearly to the architect or the underwriter a specific form of structure having beams, pillars, floor-base, partitions and walls which are incombustible.

Strictly fire-proof buildings are still rare. The name is often applied very improperly. Perhaps some few real estate agents are tempted to use the word loosely when seeking tenants; and newspapers are not always careful about exactness of words when they say that "the building burned was known as a fire-proof structure." It was said that fire-proof buildings went down as quickly as others in the great Chicago fire,—this was absolutely untrue, for there was not in the district burned over in Chicago in 1871 a single building of this class, neither was there a single fire-proof building within the limits of the

great Boston fire of 1872. Ten years ago there were not five fire-proof buildings in Boston and they were about equally rare everywhere in America. First obtaining a good foothold in Chicago about a dozen years ago the type has become more common, but the proportion of such buildings in even our largest cities is still almost infinitesimal.

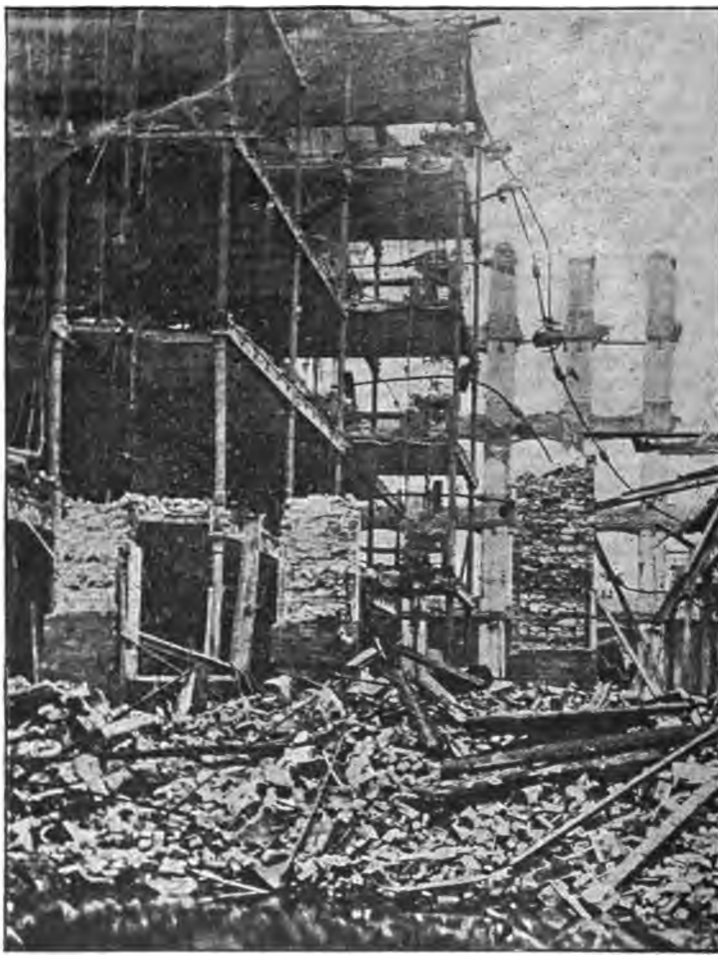


FIG. 3.—A WRECK OF A FIRE-PROOF MILL.

In England fire-proof buildings have been common for many years. A seven story mill, 140 feet long x 40 feet wide, with floors composed of brick arches resting upon cast-iron girders was successfully constructed there ninety-three years ago, and many fire-proof warehouses

and factories have been built in England from time to time, and every cotton factory built in Lancashire for about 12 years past has been of the fire-proof type, with floors of masonry supported by girders of cast-iron, wrought-iron or steel and by pillars of cast-iron.

In these English factories the cast-iron pillars and the bottom flanges of the girders have been bare and exposed to direct contact with any flame from the contents of the room, yet they have justified the name of fire-proof very well indeed. There have been many fires in fire-proof cotton mills, but when studying into the question carefully

FIG. 4.—AN OLD NEW ENGLAND MILL WITH "FACTORY ROOF."
(Nottingham Mill, Canton, Mass.)

when in England some years ago, and by correspondence since, I could learn of only two cases, among the hundreds of such buildings in existence, where fire-proof mills had been wrecked from the burning of their contents; and in both of these there was suspicion of original weakness of construction. Fig. 3 is from a photograph of one of these.

This record of so few failures among so many tests certainly justifies the name "fire-proof" and makes the outlook very hopeful for the American fire-proof structure of to-day, which is far superior to the English structure, by reason of having its iron skeleton protected by mortar or terra-cotta from direct contact with the flames instead of naked as in the ordinary English fire-proof mill.

This development of the fire-proof factory in England as compared with that of the slow burning factory in America is very interesting, the fear of fire being in each case a leading motive.

There, labor and iron were cheap and timber was dear, while here the reverse was true. Therefore in England the factory came to be built with scarcely a piece of wood from cellar to roof, except the window sash and a thin top floor of boards laid on top of the brick arched floor as being more comfortable for the feet of the operatives.

In America, pillars, beams, floors and roof were of wood. All was combustible except the side walls.

FIG. 5.—A NEW HAMPSHIRE MILL WITH FACTORY ROOF, ERECTED SIXTY YEARS AGO.
(Indian Head Mill, Nashua.)

The cost per square foot of available floor surface for the two types in their respective countries was actually about the same, although to have, in this country 5 years ago, built the English floor would have nearly doubled the cost of the mill.

So well did the American engineer learn to use his combustible material and to protect it, that the American manufacturer in a well built mill protected by sprinklers, gets his insurance just as cheaply as does the English manufacturer in his fire proof mill.

The modern tall American office building of the steel skeleton class first brought forth in Chicago and of which examples are appearing in all our great cities, is of a much superior order of fire-proofing to this English mill, because of the skill with which its iron work is shielded from direct contact with the flames. One costly accidental test of the fire resisting quality of this type of construction was had a year ago at the house of the Chicago Athletic Club and it stood the test of fire well.

The remarkably low price of structural steel within the last few months has brought about a great reduction in the cost of this type of

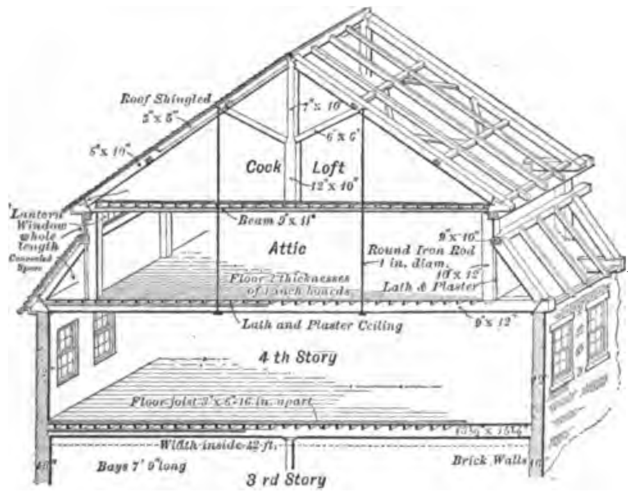


Fig. 6.—Framing of Old Factory Roof.

construction, which before was warranted only for buildings of the class where the rental is from $1.00 to $1.50 or more, per year, per square foot of floor.

Notwithstanding the recent great reductions in cost, this type of construction is still so expensive that it can be adopted only for office buildings and stores in the great commercial centers, public buildings, libraries and city residences of the most expensive class.

We have not the time now to discuss the framework or the fire proofing of this interesting class of structure; for that would of itself require a special treatise, and as students of engineering we can most profitably spend the brief time at our disposal in reviewing the evolution of the modern slow burning industrial building, studying the reasons for the adoption of some of the recognized standards. We shall find, too, that the principles of this mill construction are capable of a much wider application than they have yet commonly received.

Development of Slow Burning Construction.

The Old Factory Roof.

The whole factory system with its call for great buildings, is a development of the past century. Some few large American mills built 50 years ago are still in operation. If we study their construction we shall find them surmounted by steep lantern roofs,—the framework a heavy timber truss which supported numerous rafters of thin joist which in turn held up a roof covering of inch boards,—while over these slates or shingles were laid. Fig. 4 and 5 show the exterior of two good typical examples, and Fig. 6 shows the details of construction of the mill shown in Fig. 5.

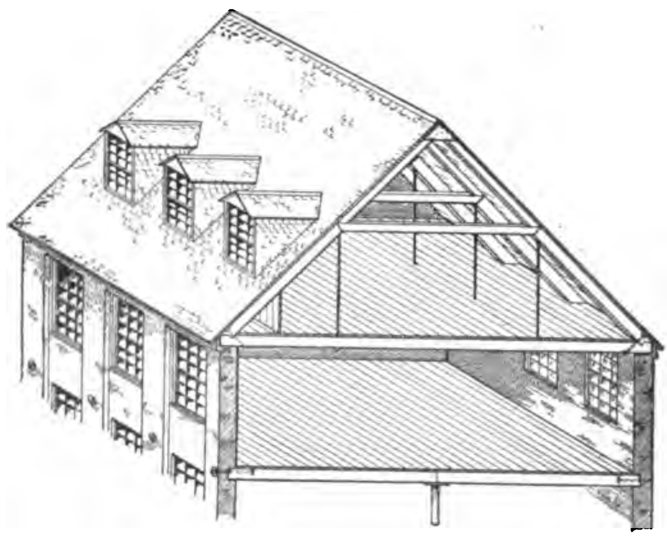


Fig 7.—The Barn Roof as Applied to Mills.

The type of roof shown in Figs. 4, 5 and 6 was the favorite one with the earlier millwrights, and is still popularly known as the "Old Factory Roof." It was made steep as the most convenient means for shedding the rain. The upper attic or "cock-loft" under it was so intolerably hot in summer, so poorly lighted, long and narrow, that it commonly served for storage of odds and ends, and in case of fire was almost absolutely inaccessible to men or to fire streams. The story next below it, the attic proper, had head room so scanty that it was inconvenient for shafting, was cramped at the sides, and worst of all was commonly built with a part of the waste room at the eaves partitioned off into a concealed space, which time and time again has proved a perfect fire trap.

The insurance companies sorrowfully paid for roof after roof of this type, and furnished thus, many a mill with the funds for rebuilding after the model of Fig. 13, page 109.

The Barn Roof.

The next era in roof building is shown in Fig. 7. This was known as the "Barn Roof."

The earlier roofs of this form, like the Factory Roof of Figs. 4, 5 and 6, were covered with inch boards on thin joist, which in turn were supported by heavier rafters, but during the transition period while more progressive mill owners were adopting the flat thick plank roof, the

Fig. 8.—Factory with "Barn Roof," (Erected 46 Years Ago.)
(Everett Mills, Lawrence, Mass.)

conservative ones fearing that a flat roof could not be maintained water tight adhered to the steep pitched roof but covered it with three inch plank. They omitted the joist and supported the plank directly upon 10 x 12 rafters 8 feet apart. If machinery was to be placed in the attic, lutheran windows were added as shown, or sometimes skylights parallel to the roof were inserted instead, as shown in Fig. 8.

The space above the collar beams of Fig. 6 was either boxed up to form a fire trap or left open and unsightly according to the wisdom of the

owners. The attic floor was hung from the roof by iron rods as shown, and the obstruction of pillars in the story next below was thus avoided. Only a little more than half the floor space is available for machinery, the light is not well distributed,—the shrinkage or settling together of the trusses often throws the floor out of level and at no time is the floor so rigid as one held up by pillars.

It was found repeatedly that a fire once started under a roof of this class was seldom extinguished until the whole roof was ruined,—that if the space at the eaves or above the collar beams was partitioned off, it shielded the fire from view and from water until it had spread the whole length of the mill, and that if the space above the collar beam was left

Fig. 9.

open, as was much safer, even then any considerable blaze at any part of the room would run up into the peak and quickly follow along from end to end of the structure, and that the weakening of the roof truss often quickly dumped the burning contents of the attic floor into the next story below and sometimes the spread of the rafters as the truss fell, pushed over the top of the side walls. The danger from falling brick or slate to a fireman on a ladder was very much greater than if the roof were flat.

Slate as a covering for a steep roof does not show the fire-proof qualities expected from an incombustible material. The black, thin slate absorbs the radiant heat so as to quicken the catching of the fire upon the wood in contact with it and with a hot fire beneath them the slates

crack off and come scaling down, sometimes inflicting ugly cuts on the firemen.

Some of these old pitched slated "barn roofs" remain on the older New England mills. Many have been burned off and many have been torn off

FIG. 10.—AN EXAMPLE OF COMBUSTIBLE ARCHITECTURE.

and the mill walls then built a story higher and a flat plank roof put on in its place, thus gaining a full story in place of an attic. Where a mill manager remodels one roof in this manner he is usually so well pleased with the change that the improvement progresses through all of his similar buildings.

Hundreds of steep mill roofs like those shown in Figs. 7 and 8 were built between 1840 and 1860, but the form proved unsatisfactory and was abandoned by the cotton mills thirty years ago. Builders of paper mills abandoned it only about five years ago, and it is still raised by many architects and "practical builders."

Illustrations of fire-trap roofs are very plenty among the works of prominent architects skilled in creating beauty of outward form. I will picture one example in Fig. 9 as recorded by my kodak this very week. It is a beautiful building, the most beautiful and expensive in the town. It contains the Public Library, the Town Hall, the offices of the Boards of Public Works with their plans and the rooms of the Town Historical Society with its valuable records. Fig. 10, shows its unused attic and the framing of its roof. At the left of the view is seen the switch board of the electric light, a pine board with pine supports joining to a roof which is as readily combustible and would be as quickly wrecked as almost anything which the wit of man could devise. The floor has large hollows with ample room for mouse nests and near the center of the end is seen a large box filled with loose waste paper, a common kind of storage for such places.

The point for your special consideration is this: The same outline and outward form could have been obtained with ten times as great endurance against fire by avoiding thin joist and plank truss work, by adopting fewer trusses, with more massive timbers, by covering with a thick plank roof and by avoiding concealed spaces in this attic floor. The difference in cost would have been so small that it could probably hardly have been found in the cost of the structureas a whole.

There was one most excellent feature common among the early American steep shingled factory roofs which may well be revived and kept alive at the present day for the cases where pitched shingle roofs appear most expedient to satisfy architectural effect as upon high-grade dwelling houses and public buildings.

This feature was the laying of the shingles over a $\frac{3}{4}$ inch bed of common lime mortar. An ordinary fire brand falling on such a roof could never burn down through, and the preserving influence of the lime is such that shingles which I have seen upon a section of roof said to have been taken from the old Allandale Mill, although deeply weathered and water-worn on top, were still fairly well preserved after a service of fifty years. It should be said, however, that there were probably better shingles in the market in those days than now.

Of course, the principal reason for the use of the steep roof from time immemorial was the ease and cheapness with which it was kept tight enough to shed rain or snow, and no doubt this force of habit

which made all roofs steep, would have been conquered earlier by the manifest andvantages of the flat roof for manufacturing purposes had it not been difficult and expensive to make the broad flat roofs water tight.

Up to about the year 1860, tin plate was the almost universal covering for such factory roofs as were built flat and it is to be noted that then any roof having an inclination so low that a man could walk over it, was called flat, or a roof with a pitch of 3 inches to the foot was called flat, whereas, to-day the standard inclination is one-half inch to the foot.

THE MANSARD MILL ROOF.

With the next stage of evolution toward the slow burning factory

FIG. 11.—MILL WITH SLOPING MANSARD ROOF.
(Atlantic Mills, Lawrence.)

roof came a roof that was in the main a flat roof. This was the mansard mill roof shown in Figs. 11 and 12.

These were applied to mills along with the general craze for this style of roof on dwellings and commercial buildings which swept over the land about twenty-five years ago. They promised well, nearly the full width of the top floor was made available and it gave promise of being much cheaper to carry this thin, wooden hip-roof up eight or ten feet and cover it with slate than to carry masonry walls up to the same height. Some hundreds of such roofs were put on large factories. The period of their introduction was brief. The floor space was somewhat inferior to that obtained with the mill roof to be next described, and the cost of the mill as a whole was found to be only a small percentage more when the masonry walls were carried to the full height of the top story.

These mansard roofs thus soon went out of fashion for mills and the great Boston fire of 1872, which leaped across streets from one roof to another of this class and progressed unchecked until 75 million dollars worth of property had been destroyed, gave the final quietus to their construction over commercial buildings.

It must never be thought that the change from one style of roof to the other was made suddenly or that the line between the periods was sharply drawn,—indeed I have seen a sporadic case of a small mansard cotton

FIG. 12.—MILL WITH FLAT MANSARD ROOF.
(Merchants' Mfg. Co., Fall River.)

mill roof put on within the past two years although as an epidemic, their construction ceased twenty years ago. Similarly the standard roof of to-day had its birth about ten years before the popularity of the mansard roof ceased.

Before leaving the subject of roofs, the Saw-tooth roof, with all its glass turned to the northern sky, should be mentioned—although these are not very common in America as yet. In my belief there is no roof or form of illumination which gives so satisfactory a light for fancy weaving or other work similarly demanding clearness of vision as the saw-tooth roof. In Rhenish Prussia in the more modern factories, the weavers of velvet and silk, nearly all work under roofs of this kind.

Fear of trouble from snow has caused them to be avoided north of Philadelphia, except in two or three instances—and those built have not embodied the principles of slow burning construction.

From studies which I have made of the problem, I believe that trouble from snow can be easily avoided, and that moreover the principles of slow burning construction may be readily applied.

The Standard Mill Roof of To-day.

The flat solid plank mill-roof of three inch pine plank covered with tin or gravel, laid upon 10 x 14 inch timbers 23 feet long, about 8 feet

Fig. 13.—A Modern Mill of Standard Construction.
(Amory Mills, Manchester.)

apart, and containing no hollows or air spaces, first came about 32 years ago and still survives in full favor with many proofs of its excellence and with every indication that the best roof possible has now been found.

In Fig. 13 and also in Fig. 2 are shown mills with this style of roof.

Simple as is this roof, it took at least a dozen years for its evolution from the flat joisted hollow roof covered with boards.

About 1850 a roof of the thin, flat, hollow, joisted type was put on the Pacific Mills at Lawrence as being the best up to date. This roof was built of boards laid on 3″ x 12″ joist, spaced two feet apart, supported on stringer timbers which in turn rested on pillars. Seven-eighths inch pine sheathing was put on the underside of the joist plank to conceal

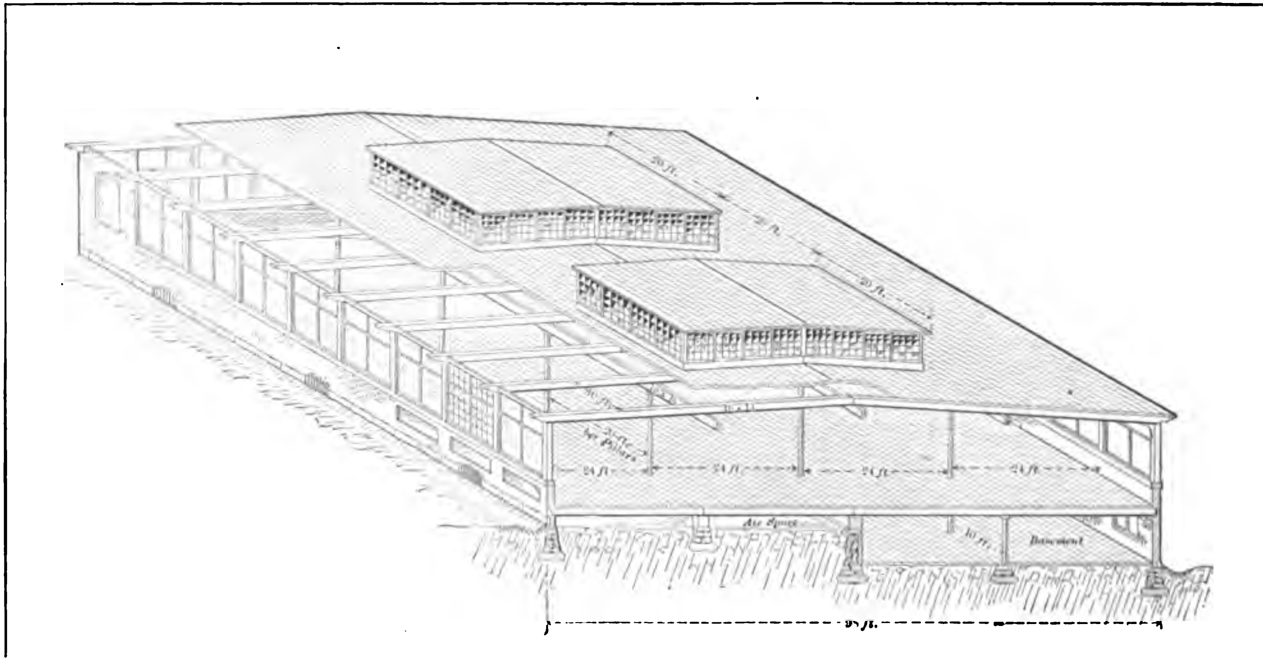


Fig. 14.—Flat Roof with Monitor Windows.

This view is presented to illustrate the great amount of window area readily obtainable in this class of construction and of value for weaving or similar purposes. Mill is of brick up to window sills, and of timber and glass above.

the unsightly appearance of the joist, but more especially to form an air space which should keep out summer heat and winter cold.

In brief this roof like many others of that period was just such a hollow roof as mill builders or architects in New York and Pennsylvania commonly put on to-day.

About 1861 or 1862 the present form of flat solid plank and timber roof was first applied to a small mill building in Manchester, N. H. It was so simple, so similar to the solid plank and timber mill floor then already popular for 15 years, that it could hardly be classed as an invention, yet the engineer who designed it has told me he had to plead hard to secure its adoption. For twenty-five years past, nothing else has been used by the leading New England mill engineers or the most enterprising managers in the cotton mill district.

Within the past half dozen years, an improvement for cold latitudes has been made by placing a one inch layer of common lime mortar on top of the three inch roof plank and then in turn placing a layer of one inch matched pine boards over the mortar. On top of these boards the gravel roof or tin roof covering is laid in the ordinary manner.

At certain Canadian paper mills in damp machine-rooms where ordinary ceilings are sure to drip with vapor condensed by the extreme cold of their winter, solid 5 inch roofs without any air space, made as just described, have given entire satisfaction, and a roof made in this way would resist firebrands thrown down on its top almost as effectually as a "fire-proof" roof of brick arches on iron beams.

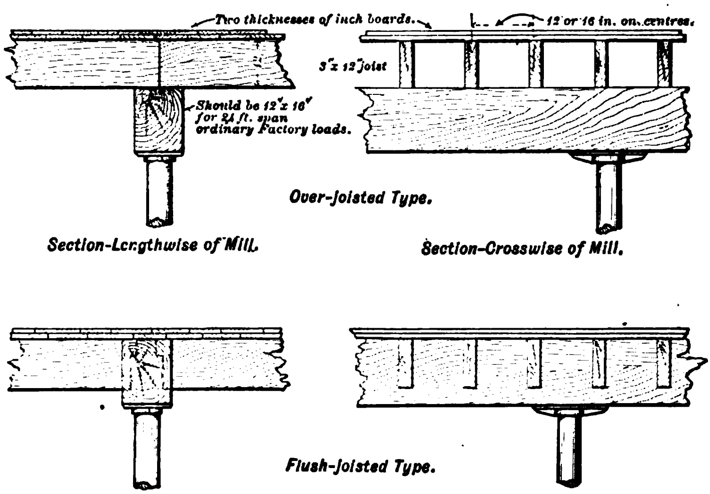
Mill Floors.

The floors of the very earliest mills were made of two thicknesses of inch boards laid on joist and ceiled up below, forming hollow spaces between,—just such a floor as many an architect would build to-day. See Fig. 6, page 102. In these hollows lint accumulated, sometimes to an almost incredible amount, sifting down in the course of years through unnoticed floor cracks, or worked in by those imperceptible air currents due to barometric fluctuations and other causes. Loose pieces of waste yarn and bits of cotton wool were also dragged here to form nests by the rats and mice that subsisted on the crumbs from the operatives' dinner pails. Oil from the machinery, water from washing the floors might easily penetrate and sow the seeds of spontaneous combustion.

A fire once started in these hollows would often work its way along and burst forth in several places at almost the same time. It was hard to locate with certainty, difficult to reach with water, and if by rare chance controlled, it was almost impossible to tell when the last spark

was quenched,* and therefore the water damage in the stories below was likely to be excessive. It was sometimes necessary to drench a factory from top to bottom to put out a fire in the hollow of a floor or roof, which a few pails full of water would have extinguished had the ceiling been open.

One of the early radical measures of the late Wm. B. Whiting and of Edward E. Manton, 30 years ago, acting on behalf of the fire prevention association already described, was to insist that all such hollow ceilings under floors should be opened completely to sight and to the application of water, by tearing off this sheathing. Mr. Whiting has told me of finding floor hollows in some old mills almost filled with lint, and of seeing cart-load after cart-load taken out from one floor. The joist and the bridging were less pleasant to view than the smooth ceiling, but they were decidedly safer.



JOISTED FLOORS.
FIG. 15.

After the earliest hollow joisted floors came the open joisted type as shown in Fig. 15, and this type of construction still survives and is quite common outside the neighborhood of the New England cotton mills. Perhaps nine-tenths of our architects or practical build-

*Even as I review these notes the morning's news tells of a fire in this city where a small fire got into the hollow roof of a piano factory. The fire was slight and with a solid plank roof would have been quenched without serious water damage, but fire being hidden in the air space holes had to be cut in the roof, two hose streams turned into them and a water damage, said to be ten or fifteen thousand dollars, was the result.

ers would still adopt exactly this, if called upon to construct a warehouse or factory to-day. It moreover, continued the type of floor commonly used in the English mills down to the time of the common adoption of the brick arched floor there ten years ago.

Experience in the textile mills has shown it thoroughly bad as regards neatness of appearance, freedom from dust and safety against fire.

Disadvantages of the Joisted Floor.

For all ordinary factory uses the solid plank and timber floor is far superior to any joisted floor, in fire resistance, neatness, and in facility for fire extinction.

The numerous thin joist can be ignited quicker than the few massive timbers just as a 12 inch pine log can be set on fire easier after it has been split into thin pieces of kindling wood.

We have already alluded to the danger of the sheathed hollow joisted floor or roof, and have spoken of the way in which inflammable lint is carried into these floor hollows by air currents and of the way in which greasy waste is dragged there by rats and mice, and if, as some experienced observers assert, rats have a fancy for nibbling at the phosphorous tips of matches, this gives one more light on the unaccountable fires which sometimes start in these concealed spaces. If a floor must be a joisted floor, then in the opinion of nearly all insurance experts, it is better to omit or to tear off the sheathing underneath the joist, even though one must then tolerate a rough and unsighty appearance, and make it an *open*-joisted floor instead of a *hollow* joisted floor.

There is in the minds of many builders a misapprehension concerning the difference in cost between the joisted, double board floor and the plank floor.

The great superiority of the solid plank and timber floor over the joisted floor has been demonstrated in many trials. The writer has seen from half to three-fourths of an inch in depth burned off from each exposed face of a 12 in. x 16 in. yellow pine beam, and the under side of the 3 inch floor plank charred into a similar depth by a fierce basement fire which could not get through this floor into the room above, although it raged from one to two hours and was so hot that it crumbled and shelled 12 inch granite pier stones into a useless heap. While wooden posts, 6 inches square, inserted midway between these stone posts to prevent vibration of the beam due certain machinery above, were merely reduced to about 4½ inches diameter by this fire and held up the load until repairs were made. No person experienced in examining fire wrecks would expect any joisted floor to have safely withstood such a

test, but there have been within the writer's knowledge several cases where plank floors have shown equal power of endurance.

Aside from such experiences and upon grounds of common sense alone, it is easy to understand the great superiority of the plank and timber floor in fire resistance. Suppose, in Fig. 16, the underside of each of the two types of floor to be charred in 1 inch deep. The 12 x 16 beam will then become 10 x 15,—still amply strong to hold up its load until repairs can be made. The plank floor too, although an inch be burned off its under side, is ample to hold up the load.

FIG. 16.

Under our joisted floor, as at the right of Fig. 16, let these 3 inch joist be charred in irregularly for 1 inch in average depth and their value as supports would cease; large pieces of floor would slump through, and the fire penetrate to the room above.

It is impossible to protect a joisted ceiling by sprinklers nearly so well as a smooth plank ceiling. With some common forms of automatic sprinklers more than half the volume of water delivered falls between the first two joist. The writer has in various experiments found the

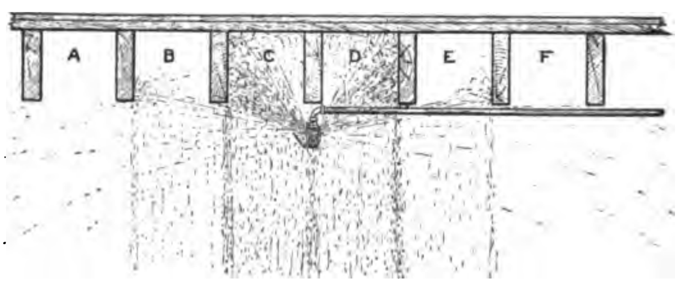


FIG. 17.

ceiling and joist surfaces at a distance of 16 inches from the sprinkler almost entirely dry after the sprinkler had been working even under 50 lbs. pressure per square inch, although in general the edges of four of the nearest joist will be wet.

With a joisted ceiling on fire, a hose stream entering a window is stopped and broken when it strikes the joist, much as illustrated for the sprinkler water in Fig. 17, whereas with a smooth plank and timber floor the stream will glide along the smooth plank surface between the beams a considerable distance, washing off every square foot of surface near where it strikes, and then deflecting will pass along much further into the room.

The earliest factory built in Lawrence Mass., forty-six years ago, was the "Bay State." These three mills were constructed by Captain Phineas Stevens, one of the most eminent millwrights of his generation. These had the "factory roof" and were about twice as high as they were wide,—indeed in them the style of high steep roofs culminated; but their floors were almost as well designed and almost as perfect as any which could be built to-day.

FIG. 18.—THE OLD BAY STATE MILLS, LAWRENCE. BUILT, 1848.

THE STANDARD MILL FLOOR OF TO-DAY.

The floor which has proved almost perfect in its fire resisting qualities, and which has in every way been found best adapted for factory use is shown in Fig. 2. It is composed of planks about 3 inches thick, sometimes 3½ or 4 inches thick, supported by Georgia pine beams 12 inches wide, 16 inches deep, and about 24 feet long, spaced 8 or 10 ft. apart and with a top floor of hardwood ⅞ or 1¼ inches thick laid over the plank and with two to three thicknesses of water proofed sheathing paper between the plank and the top floor.

There are however, some situations where the ordinary solid plank and timber floor cannot be advantageously used, notwithstanding its excellent fire-resisting qualities. For offices, schools and many commercial buildings, the drum-head-like resonance of the plank and timber floor condemns it in spite of its neatness and safety. One or two men walking with heavy boots across an upper floor will sometimes make noise enough in the room immediately below, to interrupt conversation, but it is my belief that this type of floor can be made nearly noise proof after a little experiment and earnest study.

Although our time is much too short to discuss details, we will glance briefly at the completed fire-resisting structure. The general form of the best mill-construction has already been shown in Fig. 2, page 92.

The subdivision of a plant by good fire barriers is one of the best means of increasing its safety, not only is the rapid progress of the fire prevented, but it is held within bounds such that the firemen can surround it. Fig. 19 is presented as one of many convenient ground plans and as an illustration of proper subdivision.

For a building several stories high the barriers against the fire passing up or down are almost equally important with the fire walls. A good 4 inch plank and timber floor is a most excellent fire barrier, if only it be not pierced by belt holes and elevator hatch ways. Every inside stairway, and every inside elevator, even though provided with automatic-closing hatches, is a weak spot in resisting the progress of the fire from story to story. In Fig. 19, stairways, elevators, and main belt ways are carried in separate brick shafts. Most of the best modern textile mills built within the last eight years have been designed with a perfection of "cut-off" which equals that of Fig. 19.

The principal features in the details of modern factory construction are shown in Fig. 2, page 91, and on pages 118 and 119 with such completeness that little or no further description is necessary. Of course it is to be understood width of mill, length of beam, and width of bay will vary with the requirements of different kinds of manufacture, but all ordinary purposes will be served by details of dimensions shown.

In Fig. 14, page 110, I have already sketched the Monitor roof, so called, but have made this drawing more particularly to illustrate the great amount of illumination which could be secured in a one-story mill.

The ground floor of plank laid solid on coal-tar concrete, shown at the right in Fig. 14, and also illustrated in Fig. 2, is, in my belief, nearly always possible of economical construction, and if properly built will prove as durable and as dry as one over an air space, as shown in Fig. 18, at the left. The writer would wherever possible avoid all such air spaces under floors.

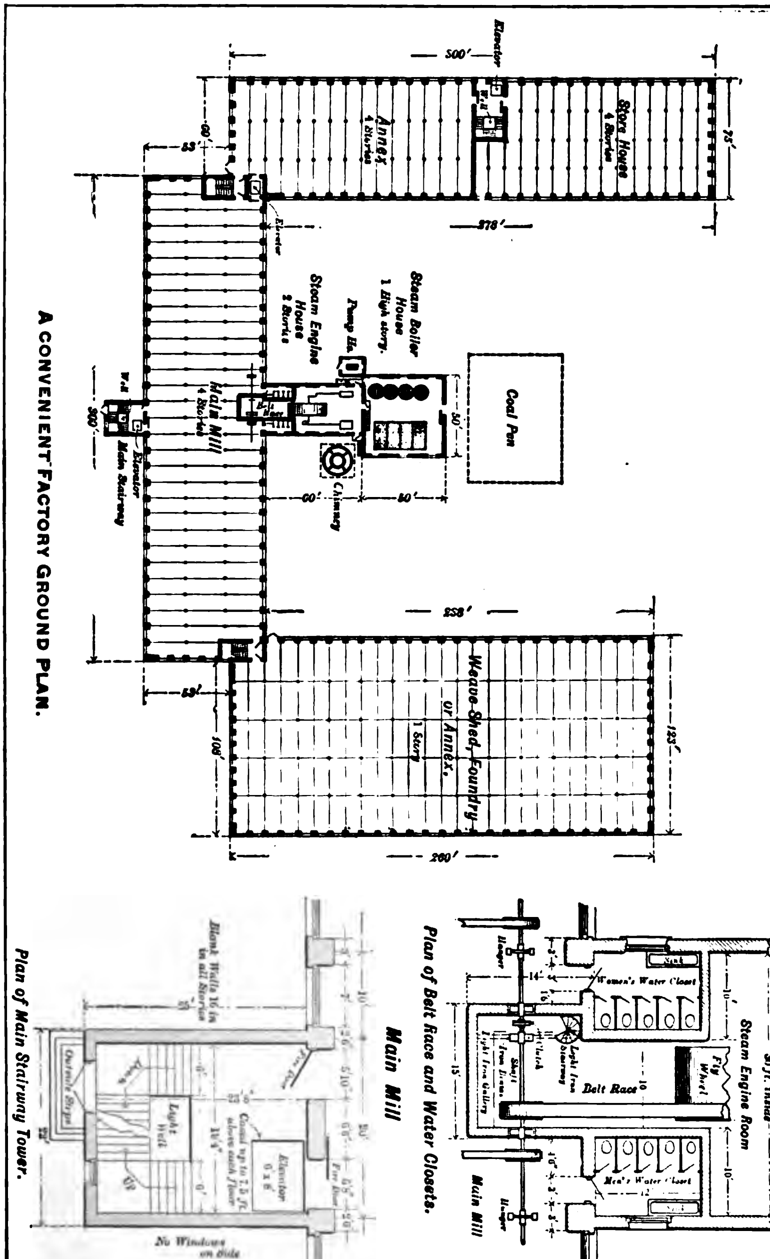

A CONVENIENT FACTORY GROUND PLAN.
Store House
4 Stories
Annex
4 Stories
Elevator
Main Mill
4 Stories
Main Stairway
Steam Boiler House
1 High story.
Pump Hs.
Steam Engine House
2 Stories
Coal Pen
Chimney
Weave Shed, Foundry or Annex.
1 Story
500′
378′
258′
260′
300′
75′
123′
108′
60′
Plan of Main Stairway Tower.
Main Mill
No Windows on Side
Light Well
Elevator
Plan of Belt Race and Water Closets.
Steam Engine Room
Belt Race
Fly Wheel
Women's Water Closet
Men's Water Closet
Sink
Hanger
Main Mill

Fig. 19.

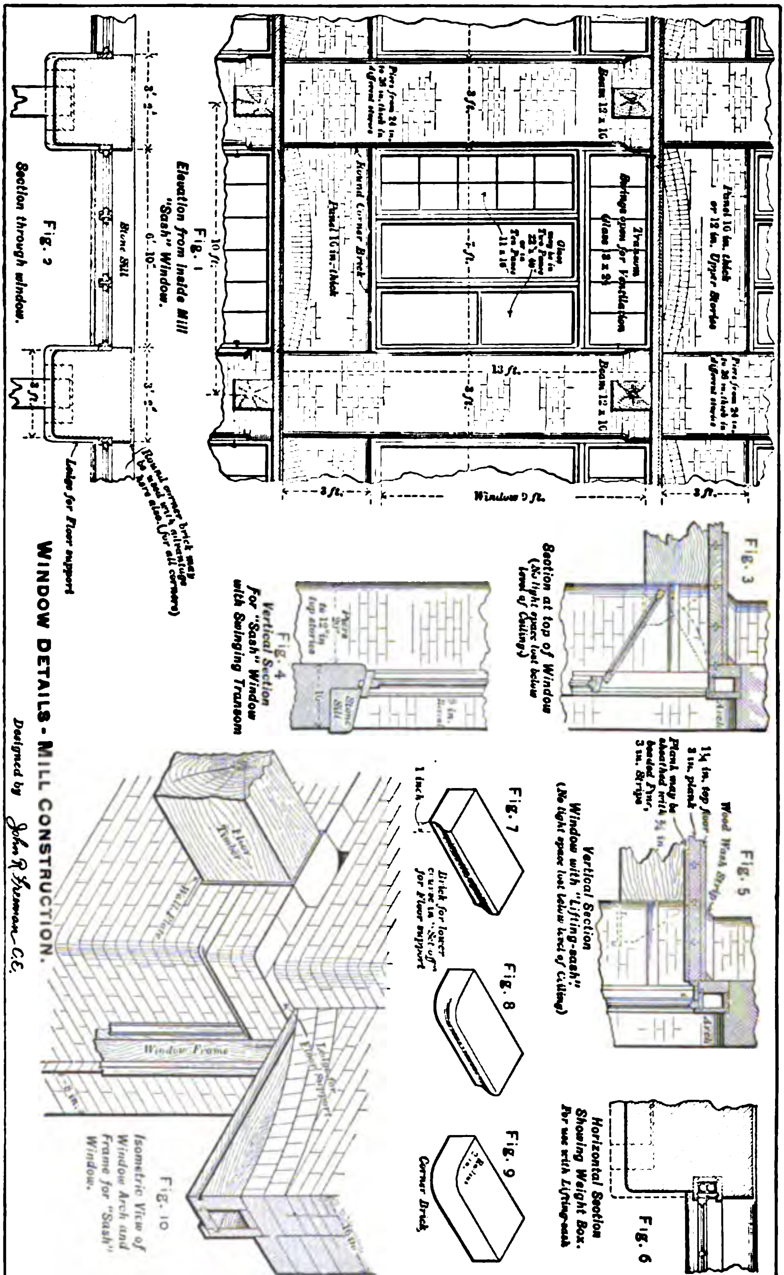

WINDOW DETAILS - MILL CONSTRUCTION.
Designed by John R. Freeman C.E.
Fig. 1
Elevation from inside Mill
"Sash" Window.
Fig. 2
Section through window.
Fig. 3
Section at top of Window
Fig. 4
Vertical Section
For "Sash" Window
with Swinging Transom
Fig. 5
Vertical Section
Window with "Lifting-sash"
Fig. 6
Horizontal Section
Showing Weight Box.
Fig. 7
Fig. 8
Fig. 9
Corner Brick
Fig. 10
Isometric View of
Window Arch and
Frame for "Sash"
Window.

FIG. 20.

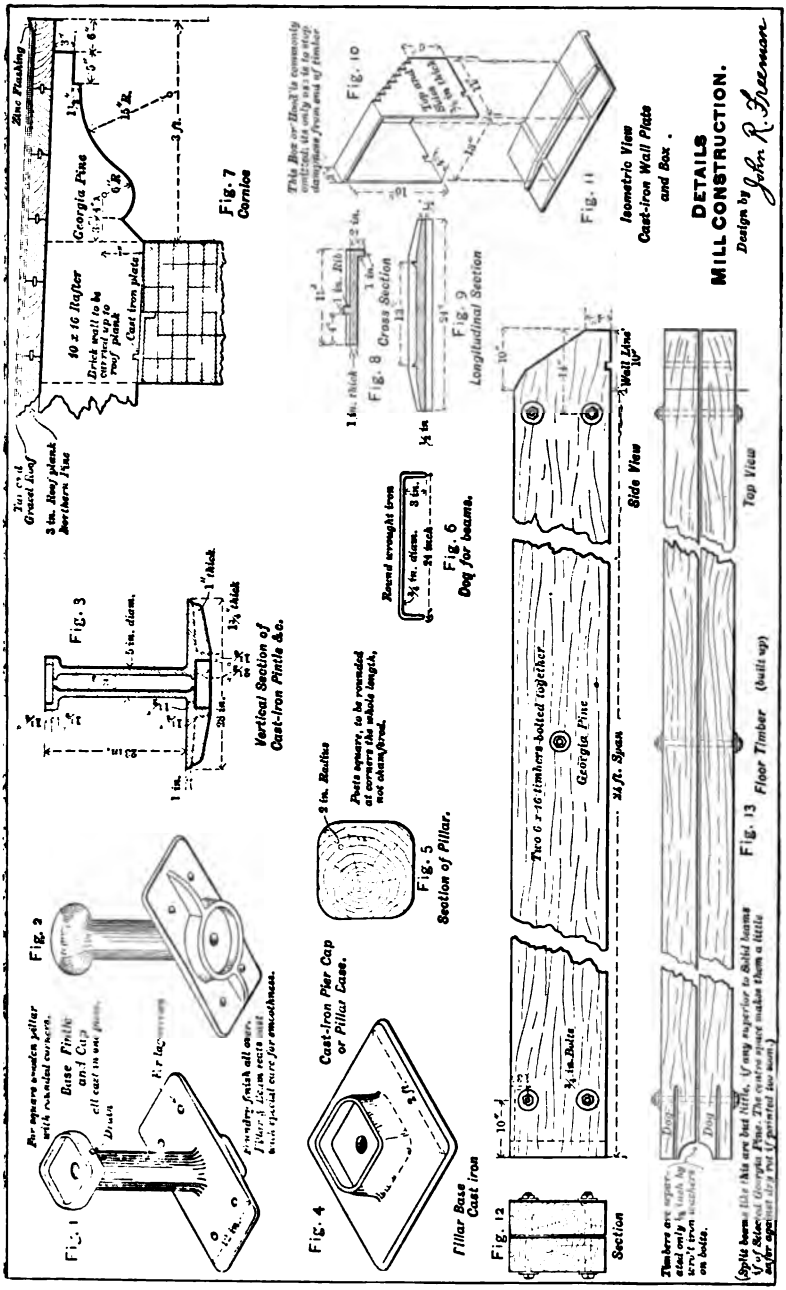

DETAILS
MILL CONSTRUCTION.
Design by John R. Freeman
Fig. 1
Base Pintle and Cap
Fig. 2
Fig. 3
Vertical Section of Cast-iron Pintle &c.
Fig. 4
Cast-iron Pier Cap or Pillar Base.
Pillar Base Cast iron
Fig. 5
Section of Pillar.
Fig. 6
Dog for beams.
Round wrought iron
Fig. 7
Cornice
Georgia Pine
10 x 16 Rafter
Cast iron plate
Zinc Flashing
Fig. 8
Cross Section
Fig. 9
Longitudinal Section
Fig. 10
Fig. 11
Isometric View Cast-iron Wall Plate and Box.
Fig. 12
Section
Side View
Two 6 x 16 timbers bolted together
Georgia Pine
24 ft. Span
Wall Line
Fig. 13
Floor Timber (built up)
Top View

FIG. 21.

Mill Floor for Extra Heavy Loads.

It has been the opinion of some good designers that only the joisted floor was strong enough and stiff enough to withstand the enormous floor loads which are in some lines of business carried upon floors above the ground level, but I believe we can show this to be a mistake.

In machine shops engaged upon a heavy class of work, the floor loads are liable to be concentrated to the extent of 300 lbs. per square foot over a whole bay, although this seldom occurs and is still more rarely a necessity. This extreme load is almost never due to the machines themselves but is nearly always due to piles of bolts, shafting or castings, in process. The writer has sometimes seen in furniture and similar wood-working establishments solid piles of oak six feet high weighing 50 lbs. per cubic foot or probably 300 lbs. per square foot of floor. Such great loads are rare, and when all is moving smoothly along, do not occur, but they are liable to occur, in places where machine-tenders are doing piece-work,—taking a piece from one pile, machining it and laying it on another pile, sometimes piling it as high as a man can reach. A few years ago I spent half a day going about the Baldwin Locomotive Works, where certain new buildings were being considered, in company with the Superintendent, searching for the heaviest second-story floor load. We found it, not in the metal working department or under the heavy machine tools, but *in the wood-working department.*

It can readily be seen that to support such loads as 300 lbs per sq. ft., the floors must be different in design from those in a woolen mill or cotton mill in which the weight of floor, stock and machinery will seldom go above 50 to 75 lbs. per sq. ft. in the most heavily loaded part of the mill. We can however, show that a good slow burning floor can be designed suitable for these enormous and unusual loads, and in fig. 22, I present a suggestion for a floor system for a heavy machine shop. The erecting floor could readily be made 10 feet wider by simple expedient without trussing.

This particular design was sketched in the effort to secure the least possible obstruction of the floor by pillars, but for all ordinary needs, if pillars are 10 feet apart lengthwise of the factory, and beams 16 feet long, the machinery can be well accommodated, and with that spacing timbers of ordinary commercial size can be made to serve, and rolled steel beams will not be need (except for crane-track).

Meanwhile I will remark that there are scores of machine shops where the ordinary mill floor, as shown in Figs. 2, 20, 21, is in use with beams of 20 ft. span, spaced 8 ft. apart, giving entire satisfaction and requiring only a little foresight to avoid high piles of heavy pieces over the middle of a beam. It can be arranged to pile the heavy loads near the

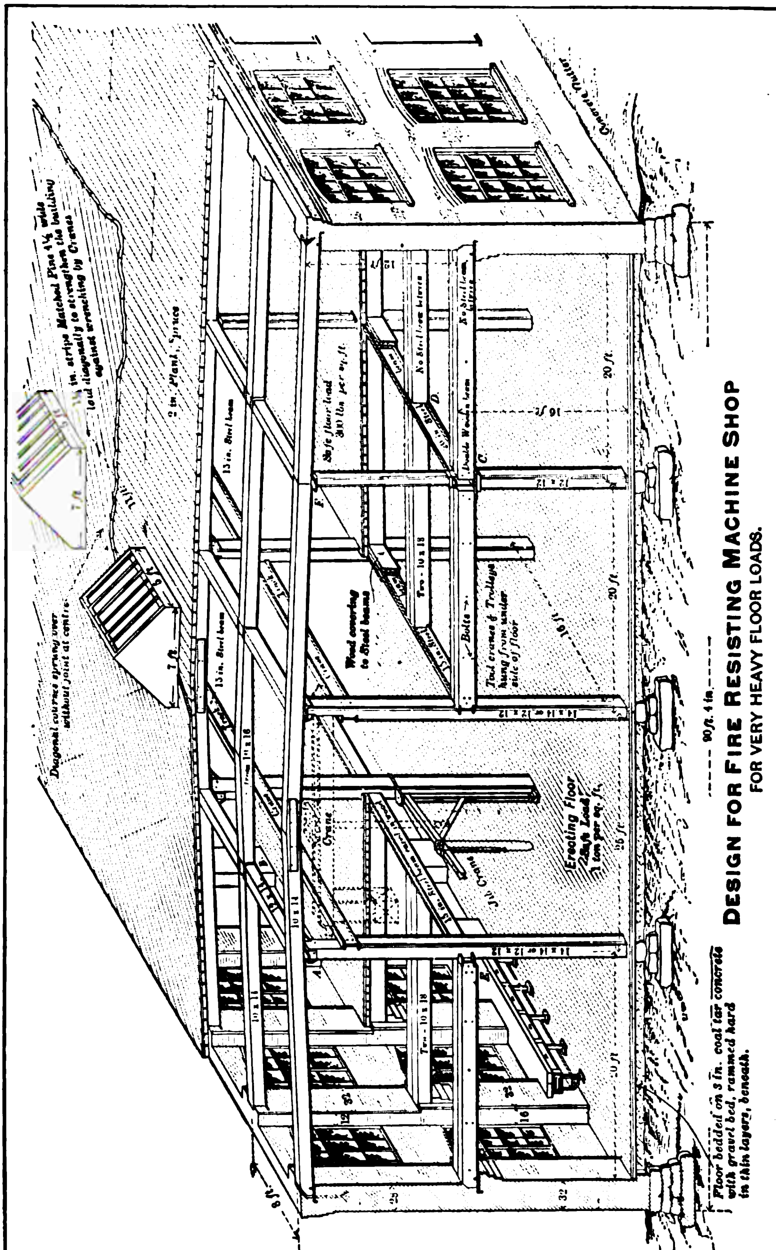

DESIGN FOR FIRE RESISTING MACHINE SHOP
FOR VERY HEAVY FLOOR LOADS.
Erecting Floor
Safe Load 1 ton per sq. ft.
Jib Crane
Tool cranes & Trolleys hung from under side of floor
Wood covering to Steel beams
Diagonal courses sprung over without joint at centre.
2 in. Plank, Spruce
Floor bedded on 3 in. coal tar concrete with gravel bed, rammed hard in thin layers, beneath.
90 ft. 4 in.

FIG. 22.

pillars or walls with little or no inconvenience, and one great element of safety in a wooden floor is that by excessive defection and ominous cracking it gives warning usually a considerable time before the limit of breaking strength is reached.

Mill Pillars.

For supporting the floors in our standard slow burning construction, wooden pillars are used in preference to pillars of either cast or wrought iron. In the first place they cost less, (say $4 for wood against $14 for iron, each for a medium size,) and in the second place they are probably somewhat safer against fire. Georgia pine is the timber preferred. Not only is this cheaper but it is stronger than an oak pillar of the same size by reason of its freedom from knots and its straightness of grain, moreover it does not check and split so badly in the seasoning.

An iron plate should always be placed against each end of a timber pillar. The sustaining power of a pillar is reduced almost one-half when it bears directly against the side of a block of the same wood, as for instance, the under side of a bolster, because the crushing strength of a large block of wood across the grain is much less than in the direction of the grain, and the cracking and spreading of the wooden bolster under a severe load starts a split in the end of the pillar when but half of the ultimate strength is reached which it will carry when tested with its end against an iron plate. A shallow ring or socket cast in the iron cap or base and fitting around the end of the wooden pillar adds materially to its endurance under an extreme eccentric load by restraining it from splitting. We may be pardoned for referring specially to so simple a detail as the iron pillar-cap since so many practical builders are to-day putting up structures in evident ignorance of the strength that is wasted when the pillar bears against wood. The writer prefers pillar, cap, pintle and base all cast in one piece as shown at Figs. 1–3, page 119, believing that more rigidity and strength in the structure as a whole is thereby secured, substantially the same form may also properly be used in connection with cast-iron pillars. The convenience of the foundryman more commonly leads to casting cap, pintle and base in three separate pieces, although the difference in cost is so very small as not to justify it.

When the height of the building is greater than four stories, cast-iron will commonly be most expedient for the lower floors, the wooden pillar for great weights becoming too much of an obstruction by reason of its larger size. Iron pillars are also best for rooms where wet work is done or where floors are damp and the bottom of a wooden pillar would be thus endangered by decay.

The fall of the Pemberton Mill (300x70x5 stories high,) at Lawrence,

thirty years ago, by which many people lost their lives and which was caused by a defective cast-iron pillar, turned the attention of our mill builders largely to the use of wood for pillars. The defect which led to this great catastrophe was that the pillar when cast, floated its core and was scarcely one-eighth inch thick on one side, while about one and one-fourth inches thick on the other side. This terrible lesson should serve to make it an invariable rule to test all cast iron pillars for uniform thickness by callipering, rolling and drilling, but it is no valid reason against using cast iron pillars, if tested and found uniform and sound.

Effect of Fire on Cast Iron Pillars.

There is a popular misapprehension as to the behavior of cast-iron pillars under fire. Newspaper accounts have led many to believe that when exposed to fire, cast-iron will crack, "become rotten" or will "fly like glass" if struck by a stream of water from the fire hose. Many underwriters have come to believe this and to think that any exposed iron work is a dangerous structural material. Some of the stories might almost make one distrust cast-iron as a material for furnace grate bars or for cook stoves, but they are largely untrue. The cast-iron pillar is often a most useful member and ought not to have its character thus defamed.

After the great "Thanksgiving Day fire" in Boston in 1889, I spent much time while the ruins of the Ames Building were being dug away to learn the condition of the cast-iron pillars. The building was filled with combustibles, and of very large floor area. The heat had been intense. About fifty steam fire engines were in service, so it is fair to suppose some of these pillars were struck by hose streams while hot.

A considerable number were broken apparently by some transverse blow as during their fall or by the fall of debris upon them, a large number were almost uninjured and *comparatively very few were badly warped or showed any evidence of having yielded to the heat.* We have since had two other two million-dollar fires in the Boston wholesale district, to which I chanced to be an eye witness, and I have come away from examining these ruins with renewed confidence in cast-iron pillars.

As an eye-witness to the great fire at Lynn which burned over many acres of stores and shoe factories in December 1889, I found there also, much to disprove the supposed treachery of cast-iron; and about five years ago I had the opportunity to visit a large cotton mill in Lancashire, England, the wooden roof of which had just been burned off; and although the fire had been so hot and long continued as to burn 12-inch wooden beams half through and to soften 2-inch wrought iron shafting so it bent freely, I found that out of 60 or 70 naked cast-iron pillars in

the room, only 2 were crooked seriously by heat, and about 90 per cent. were good enough to use over again. It is fair to remark, however, that these were top story pillars and therefore not heavily loaded.

Some instructive tests are described in a paper by Luehmann and Moeller before the Society of Engineers and Architects of Hamburg in 1888, of which an abstract by the late E. Gottlieb appears in the Journal of Engineering Societies (U. S.) for 1892, p. 85.

(These results with some of tests by Bauschinger are discussed also by Prof. Lanza in the last edition of his Applied Mechanics.)

Cast-iron and wrought-iron tubular pillars from 5 to 6 inches in diameter, about ⅝ inch thick and of 3½ ft., 6½ ft., and 13 ft. approximate length, were tested in a horizontal testing machine of 220 tons capacity. Samples were first tested cold and then other similar pieces were subjected to crushing load while heated to redness by fuel in crates placed under the pillar and by piling wood on its top. A load of from 32 to 50 tons, which is much more than a pillar of this size would be called on to bear in practice, was applied to the red hot pillar and then cold water was dashed over it in a smart stream. It was meanwhile examined for cracks, then the heating was resumed and the pillar tested for destruction.

The results, although not conclusive, regarding effect of eccentric loads and transverse strain on hot cast iron, are so interesting in correcting the popular error regarding cast-iron exposed to fire, that I have thought it worth while to collect a part of them into tabular form and present them on the following page.

I would not for a moment have it understood that I consider it satisfactory to leave the iron work of a fire-proof building exposed to the quick heat of the flames, but would maintain that for ordinary purposes of mill-work cast-iron pillars do not add to the fire hazard. Wrought iron pillars do add to the fire hazard.

The pillars of a building which is to be as nearly as possible fire-proof, or an expensive structure like the ten story steel skeleton office buildings, should have all of its iron work shielded by terra cotta or some equivalent, not merely to prevent its losing its strength, but to guard it against sudden temperature strains in case of fire. It is a good practice to fill any hollow iron pillar with a concrete of cement mortar mixed with broken stone, and costs very little.

Wrought-iron does not stand fire nearly so well as cast-iron. According to my observation a wrought-iron roof truss is twisted, warped and festooned all out of shape and beyond repair by a degree of fire which would merely char the timbers and plank of a wooden roof into a depth of half an inch. Within the past two years our inspection de-

Abstrac

STRE

Test No.	Length. Feet.	Form.	Outside Diameter Inches.	Thickness Inches.
1	8.2	Hollow, Cylindrical	5.9	0.59
2	"	"	"	"
3	"	"	"	"
4	6.5	"	"	"
5	"	"	"	"
6	"	"	"	"
7	"	"	"	"
8	"	"	"	"
9	"	"	"	"
11	"	"	"	"
12	"	"	"	"
10	"	Square, Riveted,	4.5	0.5
13	"	"	"	"

* Pillars No. 10 and No. 13
To induce cross bending
Each pillar was kept unde
the pillar: No. 2, 50 tons; No.
53 tons; No. 12, 64 tons; No. 13,
According to Francis' tab

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a de

partment has had experience of two different cases of iron roof trusses falling a tangled mass over a fire which would probably have left a plank and timber roof on wooden pillars standing.

WALLS.

Common brick burned hard is beyond all further question the best material of which to build the walls of a fire resisting building.

FIG. 23.

Brick does not disintegrate under heat like limestone and some sandstones,—does not chip and crack with exposure to fire like granite, and until pushed over by falling roof trusses or floors, will endure almost any degree of heat which a burning building can produce.

If a brick wall is built thin or with great length and height, unbuttressed by any cross wall or porch, and thus depends upon the floor ties

to steady it and hold it upright, it will naturally tumble when those supports are burned off. But I have sought in vain in the wrecks of many fires for a reasonably thick and well stiffened brick wall which has failed from the effects of fire against the bricks, and on the contrary I have seen many instances of marvelous endurance of a 20 inch or a 24 inch party wall five stories high, against one side of which the fiercest kind of a fire has completely consumed a large warehouse filled with combustibles, while on the other side the property was unharmed.

That there is a limit to the non-conducting power of even a 20 inch brick wall was shown at a fierce fire in the dry goods district of Boston a year ago. The wooden wainscoating shown in Fig. 23 was ignited by the heat transmitted through the wall from the wreck of a large five story building which lay burning fiercely in close contact with the other side of this wall for several hours, but the burning of this wainscoating was promptly discovered and extinguished by an automatic sprinkler and no person knew anything about it until the next morning.

Fig. 23 is from a photograph and is a good object lesson to teach the value of sprinklers, and that important fire walls against which combustibles may lie, should be thick, and should be laid up with an air space at the centre, but of course with careful bonding across this air space to preserve its strength.

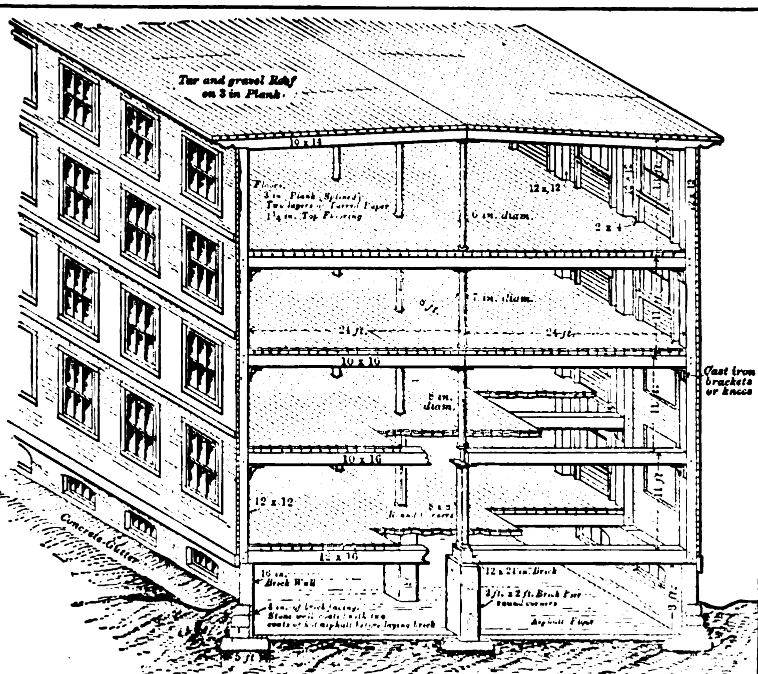
Bare walls should be the invariable rule in any industrial building,—wooden wainscoting should never be applied. In my experience in the service of the underwriters I have seen at least one striking instance of increase of water damage and great difficulty in putting out a fire which in some unknown manner caught behind a hard pine sheathing which was furred off only $\frac{1}{2}$ or $\frac{3}{4}$ inch from the brick wall. The mill owner waited for no outside advice before he had all similar sheathing torn off in that building, and told me that bare well painted brick work now looked much handsomer to him than hard pine wainscoting. If a smooth finish is desired, let it be plastering applied to metal lath and with never more than a $\frac{3}{4}$ inch space behind the lath, which thin space will be mostly filled by the mortar pressed through the meshes of the lath, but still retard dampness from penetrating.

That a bare brick wall well laid and pointed, can look neat is shown by example in watch factories, offices and silversmith works, as well as in hundreds of interiors in textile mills.

Wooden Walls.

There may be often good reason for adopting thin wooden exterior walls, as for structure needed quickly in winter, a temporary structure, or a building whose cost must be kept down to an absolute minimum.

If our building is to be but a single story in height, and if entirely free from exposure by surrounding buildings, a building can, with proper precautions, be built with wooden walls which may be regarded as equally safe against fire as a brick building, and there are to-day in existence, one story buildings wholly of wood even 300 feet long by 100 feet wide, built according to the general plan already shown, *i. e.*, with a shell of plank and heavy timbers instead of boards on joist, and which by reason of standing with a clear space of an hundred feet or more around them, are insured just as cheaply and, until other structures are built to hazard them, are very nearly as safe as though built of brick.



SLOW BURNING WOODEN (SHOE) FACTORY.

FIG. 24.

If one must build a wooden factory several stories in height, or if from motives of first cost or rapidity of construction in cold weather, wood must be used, then it is possible to so construct it that it will be far safer against fire than the average factory as commonly built heretofore. In figs. 24 and 25 I have made an essay in the application of the standard features of slow burning construction to a shoe factory.

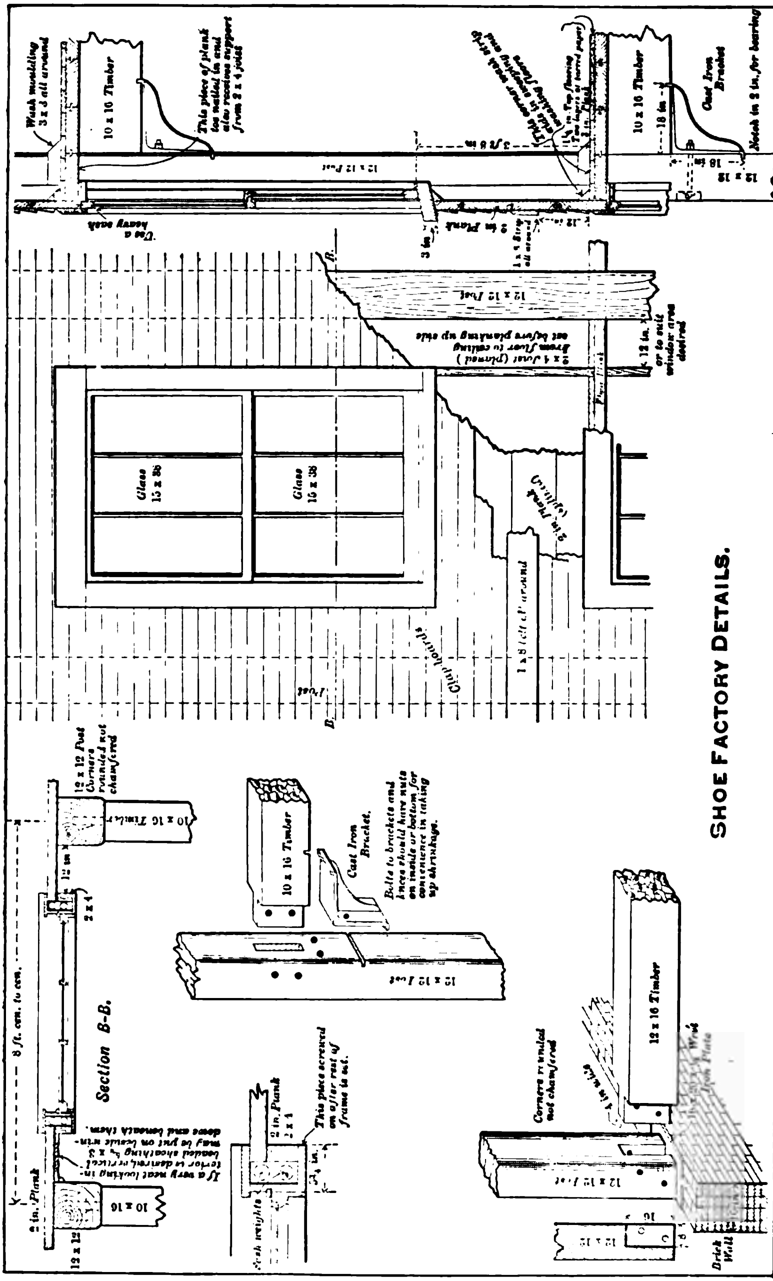


FIG. 25.

Corrugated Iron.

To protect the wooden shell of a structure already built, corrugated iron will seldom be wisely used. It is less desirable than tin as a covering, because:—

1st. It is more expensive.

2d. It absorbs and transmits heat a little more readily to the wood beneath it.

3d. When expanded by the heat of a fire it tends to cockle and warp so badly as to pull out its fastenings and let the fire in behind it.

4th. The corrugations give free access of the air to the wood behind it and it thus fails almost wholly of serving the purpose so well served by the tight fitting coating of tin, in smothering any sparks on the surface of the charred wood under it.

An experienced underwriter states that the numerous temporary wooden buildings covered with corrugated iron put up in the "burned district" in Boston immediately after the great fire of Oct., 1872, proved very disastrous fire risks; so much so that he concluded that a wooden shell was as safe without the iron covering as with it.

For warmer latitudes, the best rolling mill or furnace building which can be built, has walls and roof of corrugated iron sheets, strongly secured to light purlins of angle iron, and these in turn carried by an iron frame. If corrugated iron is used for a warehouse exposed to the burning of an adjacent building, radiant heat enough can be absorbed by the rough dark surface of these thin iron sheets to set fire to fibre or woodwork resting against their inner side, and if, as is more common in corrugated iron buildings, the purlins be of wood, the heat transmitted does often set fire to them, and with but moderate heat the sheets warp and twist so as to pull the nails from the wood.

For the New England or New York climate, corrugated iron buildings are too cold for anything but storage sheds.

Fire Doors.

The fire-door and the fire-shutter which has for the past dozen or twenty years come to be regarded as the best, is built of wood and covered with tin plate.

Doors of plate iron were formerly popular, but even when made with stiff angle-iron frames, they were found to warp badly under fire and to sometimes twist away from the seat so as to open broad cracks through which fire might readily pass.

The standard fire-door of to-day is made from a rather cheap grade of common white pine, and for the ordinary kind of door as 5 ft. x 7 ft. should be about 2 inches thick. Every portion of the door, edges and both

sides, is covered with tinned iron plates inserted with locked joints and each sheet nailed under the locking in such manner that the nail passes through both sheets of tin.

A swinging door which shuts into a 3 inch recess in a brick jamb, will give the greatest endurance, and the tightest joint for stopping smoke and sparks;—a door sliding on an overhead sloping rail can be most readily provided with a device for automatically closing itself in case of fire and is in a majority of cases, most convenient to install.

The virtue of tin plate as a fire-proof covering lies wholly in the fact that this thin sheet of iron or mild steel keeps the combustible surface behind it away from contact with the air. The tin on the steel plate is, of course, easily fusible and commonly drips off when such a door is exposed to a hot fire, and it is of use only to prevent rusting out of the sheet beneath it or to in some measure reflect back the heat and prevent its absorption.

The heat is of course quickly transmitted to the wood beneath and the wooden surface chars. Charcoal is an excellent non-conductor as is also the wood itself, and it is rare that even a very hot fire will char the wood to more than ½ inch in depth. While this charring is going on, gas is being distilled from the wood, which operation dissipates some of the heat, but more important, the little air between the wood and the tin is driven out or exhausted so that there is nothing remaining to support the combustion of the wooden surface inside the tin. We thus see the importance of not providing an air space between the tin and the wood, but on the contrary, that it is important to exclude the air, and a consideration of the foregoing teaches us that broad, thin wooden partitions tinned on only one side are likely to prove disappointing as barriers against a severe fire.

The large amount of gas distilled during the charring of the surface of the wood in a fire door or shutter must find vent or it will bulge out the tin covering and pull out the nails which hold it on. Therefore the joints between the sheets of tin should not be soldered although the solder would soon melt under a fire which would char the wood.

The nails which hold the sheets of tin should be about 1½ inches long, for if short nails or tacks are used, these will be "drawn by the heat," *i. e.*, the heat conducted in along the nail will char the wood slightly where it bites the nail and thus loosen the grip so that the twisting and expanding of the heated sheets will pull the nail out. Where sheets are not lock jointed and are merely nailed on with short nails having heads exposed to the fire, this loosening of the nails is especially likely to occur.

A door already constructed with sheets not lock jointed, should have

a clout nail at the corner of each sheet put completely through the door and clinched on the opposite side.

Although the standard door will resist the ordinary fire, there is a limit to their endurance under very hot and long continued fires. Tin covered shutters or doors commonly fail by the curling back of one corner of a sheet, thus exposing the wood to the air. In some rare instances I have seen fires so severe that the whole mass of wood inside the tin plate shell

FIG. 26.—FIRE SHUTTERS DESTROYED—FIRE HELD IN CHECK BY AUTOMATIC SPRINKLER.

was reduced to charcoal, when of course the door lost its stiffness and collapsed.

Fig. 26 is from the interior of a wholesale warehouse where the heat from an adjoining building was so intense as to destroy the shutters, but the automatic sprinklers then responded and resisted the invasion.

I have witnessed other cases where fire-shutters have safely withstood most terrible ordeals.

For the proper protection of an opening in a very important fire wall, there should be two fire-doors, one on each side of the wall.

FIRE-PROOF PAINTS.

Fire-proof paints may here be mentioned just to say that as a result of tests and from the nature of the case, the writer believes there is

nothing in existence which deserves the name. A coat of common lime whitewash probably equals the best of the so called "fire-proof" paints in fire resisting powers. Although lime whitewash on exteriors soon washes off and thus loses its value, it is often an excellent safeguard for interiors of buildings built with roughly sawn lumber and where sparks are flying.

The reason why it is almost hopeless to expect to find a good fire-proof paint that will retard ignition of a broad wooden surface when in contact with flame or exposed to strong radiant heat, is clear when we watch such a surface as it begins to char. Any impervious paint, however adherent to the ordinary surface, quickly blisters as the air contained in the wood is expanded so as the gas is distilled when the wood begins to char. A coating of paint must necessarily be too thin to be of material use as a non-conductor and the wood quickly shrivels on the surface, checks up with fine cracks and as soon as air can enter freely the wood will burn.

Until somebody devises a paint which will form a tight skin almost as tenacious as a sheet of tin, "fire proof" paints must be set down as feeble fire retardents, capable only of warding off an occasional spark.

The Protection of an Existing Factory.

Passing now to the question of how we may reduce the fire hazard at an industrial establishment whose structures have been erected and extended to meet the demands of a growing business and with little view to fire prevention, a course which will greatly lessen the danger may be outlined as follows:

1st. Enforce neatness and discipline. Slovenly house-keeping is the worst of all sources of fire. Open up all concealed spaces so far as practicable.

2d. Study to subdivide the property by fire walls or by bricking up unnecessary openings in walls and providing necessary openings with fire-doors. In a large establishment with connecting buildings it will commonly be found possible to divide the value into several groups without injury to convenience or to economy of manufacture so that though one may be burned, the others may be readily saved, and so that after this subdivision only a fraction and not the whole value will be hazarded by any one fire.

3d. Scatter frequent fire pails throughout, providing from one to three ordinary cheap 10-quart galvanized pails to each 1000 sq. ft. of floor. The records show that more mill fires are extinguished by pails than by all other means combined. Hand Grenades, Chemical Extinguishers and patent annihilators are rubbish as compared with water in a common pail, which every man, woman and child instinctively knows

how to use. Fire-pails are the most inexpensive of all forms of fire protection.

4th. Installation of Automatic Sprinklers.

5th. Private hydrants in the yard with a few lines of hose kept connected ready for instant use.

6th. A private fire pump to form a second source of water supply, although there be an excellent public supply already in the street.

7th. A private fire brigade, organized from among the operatives, drilled once a month during warm weather, and each member paid a dollar each time he is called out for fire drill. This makes membership viewed as desirable and satisfies the men for the occasional duckings they are sure to receive.

8th. Each autumn before steam heating of the rooms begins, carefully examine all steam pipes to make sure that none are in contact with wood and throughout the year, from time to time, examine all steam pipes which are alive.

Fig. 27 was from a steam pipe caught in the act of setting a wooden partition on fire.

FIG. 27.—WOOD SET ON FIRE BY STEAM PIPE.

HYDRAULICS.

We have only brief space left to devote to the hydraulics of fire protection, but it is a subject so full of interest and practical value that a hundred pages might well be written thereupon.

Water is for all ordinary purposes the only practical fire extinguisher. Carbonic acid gas, sulphurous acid gas and ammonia gas have each been the subject of numerous patents and each will work nicely in a little laboratory experiment, but a little either of theoretical computation or of plain common sense, will convince one of their impracticability elsewhere than in a mine or in the hold of a ship, or in a well planned experiment.

Steam jets once were thought worth introducing into the more dangerous rooms of factories, but have been wholly abandoned for such use in recent years.

In a conflagration two processes are nearly always going on:

1st. The roasting out of gas.

2d. The burning of the gas in a flame.

A fire cannot be extinguished by wetting the flames. The water must reach the glowing coals, cool the ignited surface to a point below which the evolution of gas will stop and then the flames will die down.

A hose stream should be so large and powerful that although half its water is turned into steam as it passes through the flames there will be water enough reach the glowing coals to quench them.

Streams smaller than 1⅛ inch are of little value on a severe fire.

Fires in structures where there is no concealed space, starting during working hours, from friction or matches or any of the ordinary causes, can nearly always be extinguished by fire-pail or garden hose if these are handy, and time may justly be said to be worth a thousand dollars a minute at the beginning of a fire in a large factory or large warehouse.

Automatic sprinklers are to be viewed chiefly as an adjunct to the night watchman. They are indeed each an ever present watchman who never sleeps, and more than 19 times out of 20, if provided with a suitable supply of water and not interferred with by lumber piled on hangers or by benches or boxes beneath which they cannot reach, will check any fire in its incipiency and prevent a small fire from becoming a great fire.

If a fire is spread by a gas explosion, if it runs along where it is shielded from the sprinklers, as within a mule carriage or within a hollow floor, until it bursts out with large volume of heat and flame, the sprinklers with pipes of size heretofore common, cannot always be relied upon to control it.

Then is the time for heavier apparatus, for the hose streams with nozzles not less than 1⅛ or 1¼ inch bore, fed by a hydrant pressure of 80 to 100 lbs.

Automatic sprinklers, as is already well known to you, are placed from 8 to 12 feet apart beneath the ceiling, and average one to each hundred square feet of floor, in rooms of ordinary hazard.

They are fed by pipes branching from a main artery and which gradually reduce in size as they pass by the tees into which the several sprinkler heads are screwed.

In the early days of automatic sprinkler protection the main artery was run down along one side of the room and the lateral branches led off crosswise of the room midway between the floor beams. To-day, central distribution is more common, but is not yet insisted upon as it would be if its merits were more clearly understood. In this system the main artery runs lengthwise, down the middle of the room, and the laterals branch off from each side, the most distant head being not more than 60 feet from the central artery.

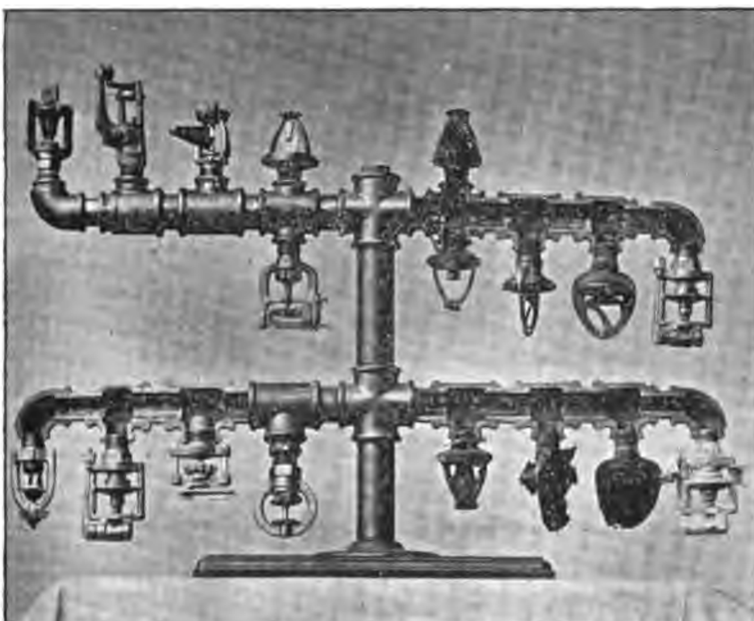


Fig. 28.—A Group of Automatic Sprinklers.

The advantage is that the end sprinkler of each line is thus brought nearer the main artery, and if a quick flashy fire opens a considerable number of heads on one line, the water does not nearly all run out at the nearest openings and leave but a feeble inadequate pressure and a little dribbling stream at the sprinklers near the end of the lateral. At the same time, central distribution is considerably cheaper.

The only objection to it is a fear that this large pipe running down the centre of the room may be in the way of belts.

I have just intimated that sprinkler pipes of the sizes heretofore common, limited the efficiency of the sprinkler. This is a question which I have recently investigated experimentally and the following table is presented to make the matter more clear. The values given below are

Number of Sprinkler From Downstream End.	Diameter of Pipe Just Up Stream From Sprinkler.		With New Clean Pipe.				With Pipes Old and Rusty.			
			Actual Pressure Found by Experiment at This Sprinkler.		Gallons per Minute Discharged by Each Sprinkler.		Computed Pressure at this Sprinkler.		Gallons per Minute Discharged by Sprinkler.	
	Present Schedule, Inches.	Proposed Schedule, Inches.	Present Schedule of Sizes.	Proposed Schedule of Sizes.	Present Schedule.	Proposed Schedule.	Present Schedule.	Proposed Schedule.	Present Schedule.	Proposed Schedule.
No 1 End	$\frac{3}{4}$	$\frac{3}{4}$	2.50	2.50	8.6	8.6	2.50	2.50	8.6	8.6
2	1	1	3.00	3.00	9.2	9.2	4.1	4.1	10.8	10.8
3	1	$1\frac{1}{4}$	4.3	4.3	11.0	11.0	5.5	5.5	12.6	12.6
4	$1\frac{1}{4}$	$1\frac{1}{2}$	7.8	4.5	15.0	11.4	11.8	7.4	18.8	14.7
5	$1\frac{1}{4}$	$1\frac{1}{2}$	8.5	4.8	15.8	11.7	14.2	8.6	20.6	15.9
6	$1\frac{1}{4}$	2	10.4	5.7	17.4	12.9	18.2	11.1	23.5	18.2
7	$1\frac{1}{2}$	2	14.2	6.3	20.7	13.5	29.8	12.1	31.0	19.0
8	$1\frac{1}{2}$	2	16.4	6.6	22.2	13.8	37.	13.4	34.3	20.0
9	$1\frac{1}{2}$	2	18.2	7.0	23.4	14.3	55.	15.2	42.0	21.4
10	$1\frac{1}{2}$	2	23.0	7.5	26.4	14.8	76.	16.1	49.6	22.1
11	2	$2\frac{1}{2}$	30.2	9.4	30.7	16.6	114.	24.8	. .	27.7

derived from experiments with the Grinnell sprinkler; other forms would give results but slightly different.

2½ pounds per square inch or about 6 feet head measured at the sprinkler while water is flowing, is the very least that will scatter the water suitably or that will force out a proper volume. The table therefore starts with this minimum pressure of 2½ pounds on the end sprinkler in every case,—it will be seen that with the present schedule of pipe sizes 23 pounds pressure would be necessary at the other end of the line in a mill 100 feet wide when pipes are new, or 76 lbs., after pipes become rusty, would be needed to have the end sprinkler do good effective work, supposing in each case that the main artery ran along the side of the room. From this table too the superior merits of central distribution can be easily shown.

In the larger pipes beyond the limits covered by this table, I have not proposed any great change in number of heads allowable, In cases where central distribution is used, the proposed schedule will impose but an increase in cost, which is just about equal to the recent reduction in the cost of iron pipe and fittings.

In perhaps 9 out of 10 fires, the sprinklers operate so promptly that less than 10 heads have opportunity to open before the fire is controlled, and as these ten heads will be upon three or four different laterals, the old pipe sizes will prove all right. If however, 50 or 75 heads be open, then only part of the sprinklers will be fed properly, some near the main artery will waste the water under excess of pressure; those distant, near the end of the branch will be without water, and this fact I believe explains some of the cases where buildings fully equipped with sprinklers have been destroyed.

Hydrants.

If we had time it would be interesting to present at length the considerations which should govern the installation of a hydrant system for a factory, and the larger problem of a proper arrangement for the hydrants and water mains of a city, so that the fire department may be properly served.

We have only time to remark that one good fire stream discharges a number of gallons per minute sufficient to supply the domestic needs of about six thousand inhabitants.

That a four inch pipe is too small to ever be used for a hydrant main. Six inch pipe should be smallest ever laid for a hydrant pipe.

That arranging a system of pipes for fire service of a city or a mill-yard is not so much a system for water *distribution* as it is a system for *concentration* of water.

That instead of arranging the pipes like the branches of a tree with the trunk in the centre, the main arteries had best be *along the margins* with cross connections between, and that it is true economy to be generous in the number and frequency of hydrants, and save the yearly expenditure for hose. It appears reasonable to say that in the average village and city the hydrants are just about twice as far apart as true economy would dictate.

Hydrants or their feed pipes sometimes freeze, a hydrant valve sometimes sticks. If any one hydrant is found unavailable in the minute of need there should be another within from 250 to 400 ft. from which the hose can be run. 100 ft. of 6-inch pipe laid 5 ft. deep, all complete, costs no more than 100 ft. of the best 2½-inch fire-hose. The pipe will be in servicable condition after 50 years; the rubber lined hose will commonly be worn out or rotten at the end of 5 or 10 years.

Water for a good stiff 1⅛ inch 250 gallon stream cannot be forced more than 400 ft. through 2½-inch fire-hose unless the public water main gives a head of more than 230 ft. or 100 lbs. The average water works pressure is about 75 lbs., therofore if any one would use hose lines more than 300 ft. long he must sacrifice in volume of force of the jet, or must have a steam fire engine in the line to give the water an extra push.

A 250-gallon stream has of late been adopted by many as the standard fire stream. A small jet may all be turned into steam as it goes through the flames, while with a large jet though half will be evaporated enough may be left to reach and quench the glowing coals which form the heat of the fire. Given 1200 gallons of water per minute under good pressure, this will do more good on a fierce fire if concentrated into four 1¼-inch streams than in six one-inch streams. A 250-gallon stream requires 16.34 feet per second velocity in 2½-inch hose. We are held up to this high force, wasting velocity by the necessity of keeping the hose so small in diameter that it can be easily handled. Therefore, as many gallons per minute are put through a 2½-inch hose as are put through a 6-inch pipe at the 3-foot velocity which is seldom exceeded in long water mains.

Therefore hydrants should be near together that the lines of hose may be short and that a larger number of streams may be concentrated on the heat of the fire.

In a paper before the New England Water Works Association, June, 1892, the writer ventured to formulate the following rough general guide to the number of fire streams which a system of municipal water supply should be able to furnish.

Total population of community protected.	Number of 250 gallon streams which should be available in addition to the maximum draft.
1000	2 to 3
5000	4 to 8
10000	6 to 12
20000	8 to 15
40000	12 to 18
60000	15 to 22
100000	20 to 30
200000	30 to 50

Ten streams or as large a proportion thereof as financial considerations will permit may be recommended as necessary for proper protection of a compact group of large valuable buildings, irrespective of small population. As a general statement, subject under special circumstances to modification, we should recommend that the pipes be large enough and the hydrants numerous enough so that two-thirds the number of streams given in the table for the larger communities can be concentrated upon any one square in the compactly built portion of the town.

The municipal hydrant system should be designed to cope with an extraordinary fire, to overtake it and stop it even after it has got one or two hours start of the firemen, and it can be shown that a city water pipe and hydrant system can be arranged to afford this supply by a judicious location of the main arteries and by frequent gridiron connections, all at a percentage of increased expense which is not large or unreasonable.

For the hydrant system of a factory we have only space to remark, 1st. Every branch pipe of more than 2½ or 3 inches diameter, entering a building, should have an outside shut-off conspiciously located, 50 feet away from the building if the grounds will allow, by which waste can be stopped and pressure saved if the pipes inside become broken during a fire.

2d. No part of the main system should pass beneath a building lest it be broken by the falling of a wall or floor.

3d. Hydrants should be from 50 to 70 feet distant from the buildings if grounds will allow, or at least a distance equal to the height of the wall.

4th. Two 2-way hydrants are safer than one 4-way hydrant in view of frost or possible temporary derangement.

5th. There should be hydrants enough surrounding any particular building so that the full number of fire streams which the mill fire pumps and the public supply together can deliver, can all be concentrated on any one important building with lines of hose averaging not more than 150 feet in length.

6th. Over from ¼ to ½ of these, hydrant hose houses should be built, each containing at least one line of cotton rubber lined hose 100 feet long, coupled up and ready to be run out for instant use.

7th. 2½ inch hose on stand-pipe *inside* a room is seldom advisable. If a fire gets beyond the fire pails and is too fierce to be extinguished by the small hose, by that time the smoke drives the men out of the room and if hose from inside stand pipes has been laid the water is likely to be left wasting as the men retreat, thus robbing the sprinklers and the outside hydrants of their proper supply.

8th. A 6 inch pipe is the smallest which should ever be used for a hydrant main. Use 8 inch for the main arteries. In a large yard where values are great, even a 12 inch may be the most economical size for the main arteries.

9th. Use hydrants with gates and risers not less than 5 inches in diameter.

There are many more points which we might profitably discuss, did time allow; for we have only had opportunity to outline the subject briefly. I can assure you all, that the principles of fire protection afford a worthy and profitable study for one who is fitting himself to become an Engineer or a Captain of Industry.

TRANSACTIONS

OF THE

ASSOCIATION OF CIVIL ENGINEERS

OF

CORNELL UNIVERSITY

1895

TRANSACTIONS

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OF

Civil Engineers

OF

CORNELL UNIVERSITY

VOL. III. 1894–1895.

CONTAINING

ADDRESSES BY NON-RESIDENT LECTURERS, MISCELLANEOUS PAPERS, CONSTITUTION, AND LIST OF MEMBERS OF THE ASSOCIATION

NOTE.—This Association is not responsible for any statements or opinions advanced in any of its publications.

ITHACA, N. Y.
JUNE, 1895

ANDRUS & CHURCH,
PRINTERS,
ITHACA, N. Y.

CONTENTS.

ILLUSTRATIONS.

CONSTITUTION

OF THE

ASSOCIATION OF

Civil Engineers of Cornell University

PREAMBLE.

WE, the undersigned, members of the Senior and Junior classes in the College of Civil Engineering of Cornell University, do hereby form ourselves into an Association for the discussion of engineering topics, and the promotion of general information on engineering subjects, and do hereby agree to abide by, and sustain the following Constitution and By-Laws:

ARTICLE I.

NAME.

1. This Association shall be known as the Association of Civil Engineers of Cornell University.

ARTICLE II.

MEMBERSHIP.

1. The Association shall consist of Active and Honorary members.

2. All Alumni of this college and all students recognized as upperclassmen, and registered in the College of Civil Engineering, are eligible to membership in this Association.

3. Any eligible person may become an honorary member by a two-thirds vote of the members present at any regular meeting. Such members shall have privileges of active members except those of voting and holding office, and shall be exempt from all dues.

4. The membership fees of this Association for all active graduate members shall be $1.00 per annum. All money received from membership fees shall be devoted to defraying cost of publication of non-resident lectures delivered before the Association. All other expenses of this Association shall be met by direct tax upon the undergraduate members.

5. A copy of each lecture delivered before this Association shall be forwarded to each member of the Association.

ARTICLE III.

OFFICERS.

1. The officers of the Association shall consist of a President, Vice-President, Recording Secretary, Corresponding Secretary, and Treasurer.

2. The President shall preside at all meetings of the Association and enforce the Constitution and By-Laws, and shall call special meetings at the request of five active members.

3. The Vice-President shall take the chair at the request of the President, and shall act as President in his absence. The Vice-President shall be chairman of the appointment committee.

4. The Recording Secretary shall keep minutes of proceedings of all meetings of the Association and shall post notices for the same.

5. The Corresponding Secretary shall attend to all the necessary correspondence of the Association. He shall be elected from among the Faculty of the college.

6. The Treasurer shall receive all money and dues, and shall pay all bills of the Association, such bills to meet the approval of the Executive Committee before such payments. He shall make a report when called upon by the Association and also when his term of office expires. He shall be Chairman of the Executive Committee.

7. The officers shall be chosen by ballot at the last regular meeting of the spring term, from the Junior Class, and shall hold office until their successors are elected.

ARTICLE IV.

COMMITTEES.

1. There shall be two Standing Committees, an Executive Committe and a Committee on Appointments. Each committee shall consist of three members, and be appointed at the beginning of each term by the President.

2. The Executive Committee shall see that the rooms of the Association are ready for occupancy previous to all meetings, and shall transact such business as may be referred to it by the Association.

3. The Committee on Appointments shall make appointments for all literary exercises for each meeting, and such appointments shall be posted at least two days before reading. The committee shall furnish the Secretary with a list of such appointments.

ARTICLE V.

AMENDMENTS.

The Constitution or By-Laws may be amended by a two-thirds vote of all members present at any regular meeting; such amendment to be before the Association at least one week.

BY-LAWS.

ARTICLE I.

REGULAR MEETINGS.

Regular meetings shall be held on Friday of each week, in the Association rooms, commencing on the first Friday after registration week, and ending on the last Friday but one before examination week of each term.

ARTICLE II.

QUORUM.

One-third of the active undergraduate members of the Association shall constitute a quorum. No business can be transacted without a quorum being present.

ARTICLE III.

ORDER OF PROCEEDINGS AT A REGULAR MEETING.

1. Roll Call.
2. Minutes of Preceding Meeting.
3. Literary Exercises.
4. Unfinished Business.
    - *a.* Report of Standing Committees.
    - *b.* Report of Special Committees.
    - *c.* Report of Officers.
    - *d.* Miscellaneous Business.
5. New Business.
6. Adjournment.

ARTICLE IV.

EXERCISES.

The exercises shall consist of discussions, memoirs, essays, papers, lectures, and such other exercises as the Association shall from time to time direct.

ARTICLE V.

SUSPENSION OF BY-LAWS.

A By-Law may be suspended for one meeting by a vote of two-thirds of the members present.

H. R. LORDLY,
E. J. FORT,
H. D. ALEXANDER,
Committee.

OFFICERS FOR 1894-95.

President,

WARNER W. GILBERT.

Vice-President,

ALBERT H. SEABURY.

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LORIN H. IRELAND.

The President's Address.

Members of the Association of Civil Engineers:

Although it is about twenty years since the first association was formed among the civil engineers of Cornell, the present flourishing association has been organized only three years. The new association has from the start been successful and it remains with the present Juniors to continue it so.

During the past winter our Association received an invitation to send a representative to the banquet given by the students of the School of Practical Science, Toronto University. Mr. Zarbell was sent in response to the invitation, and he was very cordially received and well entertained. This invitation is the second invitation of the kind that we have received in two years, the other being from the undergraduates in science at McGill University. We have done nothing towards returning these compliments but something should be done if possible.

The civil engineering students at Cornell are benefitted, in a way that the students at few other engineering schools are, by having an opportunity to hear a great many celebrated non-resident lecturers each year. The list this year has been full, and the lectures have been very good, but I am sorry to be obliged to say that the attendance at one or two of the lectures was very small. The small attendance, was, however, the exception and not the rule.

Several of our faculty delivered very interesting lectures before the Association, the one of Prof. Jacoby's on "Indexing" being especially interesting, in that it treated of a subject seldom brought before the engineer. Mr. Comstock gave a lecture on "Steel" and Mr. Sherman one on "Fire Protection Engineering," both of which were very good. Also, a number of the members contributed short entertaining papers.

Attention is called to the Fuertes Medals founded by Prof. Fuertes. These medals, two in number, are to be awarded annually as follows: One is to be given by the Faculty to that student of the College of Civil Engineering who may be found, on graduating, to have maintained the highest degree of scholarship in the subjects of his course during four consecutive years; and the other medal will be given annually by the Faculty to that graduate of the College of Civil En-

gineering who may write a meritorious paper on some engineering subject tending to advance the scientific or practical interests of the Civil Engineering profession.

However, neither medal is to be given, if, in the opinion of the Faculty there is no candidate of sufficient merit to entitle him to such distinction.

We owe a great deal to Prof. Crandall for the manner in which he has aided us this past year, and I take this occasion to thank him.

I thank the other officers for their good work during the year, and especially do I thank the publication committee for their untiring efforts to make Volume III of the "Transactions" a success.

In closing, let me say to the Juniors, keep up your interest in the Association. Do not let it flag, but make the meetings attractive and instructive. Hold regular meetings, and attend these meetings. Every member do his best for the advancement of the Association's interests.

With the best wishes for the future of the Association, I will bid you farewell.

W. W. GILBERT.

Ithaca, May 17, 1895.

OFFICERS FOR 1895-96.

ELECTED AT THE LAST REGULAR MEETING, MAY 17, 1895.

President,

HARRY K. RUNNETTE.

Vice-President,

JOHN H. LANCE.

Treasurer,

DANIEL Y. DIMON.

Corresponding Secretary,

PROF. CHAS. L. CRANDALL.

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EMILE A. VAN CAUTEREN.

MEMBERS OF

THE ASSOCIATION OF CIVIL ENGINEERS,

CORNELL UNIVERSITY.

ALL GRADUATES OF THE COLLEGE OF CIVIL ENGINEERING UP TO AND INCLUDING 1895.

The address on the line with the name is the home or permanent one ; that on the line below is the business or usually more temporary one. When only one is given its position is an index of its permanency, except when forced from one position to the other by lack of room. All the addresses have been verified since Dec. 1, 1894, except those in italics, which are unreliable. All changes in address should be reported promptly to the Corresponding Secretary.

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DECEASED MEMBERS AND GRADUATES.

NAME.		RESIDENCE.	DATE OF DEATH.
Ames, Willis C	*C.E.*, '77 ;	Whitney's Point, N. Y.	Feb. 23, 1894
Aylen, Charles P	*C.E.*, '76 ;	Aylmer, Canada	1893
Bueno, Francisco de A. V.;	*C.E.*, '76 ;	Rio de Janeiro, Brazil	About 1881
Carpenter, Frank De Y	*C.E.*, '73 ;	*M.C.E.* '76; Highland, N. Y.	Dec. 19, 1883
Clark, Ira E	*C.E.*, '72 ;	Weston, Mass	May 23, 1882
Cook, Isaac N	*C.E.*, '75 ;	Jersey City, N. J	May 7, 1885
Cooper, Edgar H	*C.E.*, '85 ;	New York City	Oct, 1890
Dodd, Franklin, M. G	*C.E.*, '90 ;	Franklin, N. J	Sept. 13, 1891
Dobraluboff, John A	*C.E.*, '74 ;	Nijney Novgorod, Russia	About 1882
Eidlitz, Alfred F	*C.E.*, '76 ;	New York City	April 22, 1877
Farnham, Whitfield	*C.E.*, '71 ;	*M.C.E.*, '74 ; St. Louis, Mo	April 13, 1895
Fitch, William R	*C.E.*, '74;	Ithaca, N. Y	April 14, 1886
Gunner, Daniel W.	*C.E.*, '87;	Schagticoke, N. Y.	Oct. 10, 1887
Holbrook, Ernest M	*C.E.*, '89;	*M.C.E.*, '90; Ithaca, N. Y	Oct. 9, 1892
Hulse, Howard C	*C.E.*, '91 ;	Brooklyn, N. Y.	Feb. 20, 1893
Landers, Herbert H	*C.E.*, '90;	Green Island, N. Y.	Feb. 4, 1893
Lyman, George F	*C.E.*, '73;	Tenafly, N. J	Dec. 25, 1880
MacMullen, Justus C.	*C.E.*, '76;	Unionville, N. Y.	Jan. 31, 1888
Preston, Kolce	*C.E.*, '73;	Wilmington, Del.	Jan. 4, 1876
Sheldon, Daniel C.	*C.E.*, '83;	Delphi, N. Y.	Oct. 2, 1893
Shepard, Frank W.	*C.E.*, '86;	Medina, Ohio	Feb. 10, 1892
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Smith, William J	*C.E.*, '79;	Charleston, N. Y.	Dec. 3, 1886
Stewart, Neil, Jr.	*C.E.*, '87 ;	York, N. Y	March 30, 1891
Tilley, George A.	*C.E.*, '73;	Washington, D. C.	March 14, 1877
Tompkins, John H.	*C.E.*, '73;	Poughkeepsie, N. Y	July, 1879
Viegas-Muniz, Joaquim	*C.E.*, '77 ;	Piracicaba, Brazil	About 1883
Wightman, Willard H	*C.E.*, '81 ;	Ashland, Ore	Oct. 29, 1889

In Memoriam.

Willis Chester Ames.

Willis Chester Ames was born at Upper Lisle, N. Y., Jan. 29, 1854. During his boyhood he resided at home, attending the Whitney's Point Academy, where he received his preliminary education and was known as an ambitious student, as well as a genial and popular schoolmate.

He was the first of the students of that institution to obtain a Cornell free scholarship and at the age of nineteen entered the Civil Engineering course of this University. While here he took first rank as a student and graduated with high honors in the class of '77.

Shortly after his graduation he entered the Govt. Coast Survey Office at Washington, D. C., and soon gained the distinction there of being one of the most correct and efficient clerks in the department.

In 1881 an offer from the Mexican National Construction Co. drew Mr. Ames to Mexico where he has since lived, and where his services not only won for himself the reputation of an exact civil engineer, but also, in many instances, secured considerable financial gain to the companies who employed him, and whose interests he faithfully served. More recently he was chief engineer in a railroad survey on the isthmus of Tehuantepec.

During the summer of 1889 he made his last visit home and before returning to Mexico was married Oct. 16th to Miss Lulah Jennings of a well known family in Warrenton, Va.

His death occured in the City of Mexico, Feb. 23, 1894, after a brief illness.

George Whitfield Farnham,

The death of George Whitfield Farnham occurred Saturday, April 13, at St. Helena, California, whither he had gone, accompanied by his daughter, about the first of March last, hoping to recover from an illness of a chronic nature.

The deceased was a member of the class of '71 of Cornell University, and was for many years city engineer of Elmira and roadmaster of the Susquehanna division of the Erie railroad where his accomplishments were fully recognized by all. Early in 1887, Mr. Farnham was called by Major Robert M. McDowell, vice-president and general man-

ager of the coal department of the Missouri Pacific railway company (Gould system) as chief mining engineer and purchasing agent with headquarters at St. Louis. Here he sustained the high reputation already achieved in the east, both in private and official life. Words fail to do justice to one so meritorious in all that constitutes a model man. He was conscientious, brave and talented, and thus summarized, what greater tribute can those who mourn his absence bestow upon him.

WILLARD HUMPHREY WIGHTMAN.

Willard Humphrey Wightman was born at Hastings, Oswego County, N. Y., January 31st, 1852. He was educated in the public schools and Mexico Academy, after which he was engaged for a short time at engineering work.

In 1877 he entered Cornell University and took the full course of civil engineering, graduating in 1881. Soon after graduating he entered the service of the Union Pacific Railroad, where he served in various capacities in the engineering department on that road for a period of three years.

In 1884 he accepted a position on the Northern Pacific Railroad as office draftsman. In the spring of 1885 he was appointed on the same road as Resident Engineer on the construction of its Cascade Division, having charge of the grading and revision of surveys for one of its residencies. Shortly afterward he was assigned to duty as Assistant Engineer in charge of track-laying and bridging.

During the winter of 1886 and 1887 he left railroading, and was engaged in making Government surveys in Idaho and Oregon as Deputy United States Surveyor. In the spring of 1887 he again accepted a position on the Northern Pacific Railroad as Assistant Engineer and Superintendent in charge of the construction of the Kennewick Bridge across the Columbia River, near Pasco, in the then Territory of Washington, where he remained for one year.

In 1888 he was appointed Assistant Engineer in charge of the construction of the Spokane and Palouse Railway, which is a branch of the Northern Pacific extending from Marshall, Washington, to Genesee, Idaho, a distance of 106 miles. He continued to act in that capacity until a few weeks before the time of his death, when he resigned on account of ill health.

He died at his home in Ashland, Oregon, October 29th, 1889. His death was caused from taking a severe cold, just as he was getting over the measles, which resulted in lung trouble, thus carrying him off in a few months.

In all the various engineering positions which Mr. Wightman occupied he was trusted both by employer and employed. He was an engineer of no mean merit and promise, and left behind him some very creditable work. In his death the American Society of Civil Engineers lost a competent and faithful member of the profession.—*Proceedings of American Society of Civil Engineers.*

NON-RESIDENT LECTURERS

OF

THE COLLEGE OF CIVIL ENGINEERING,

CORNELL UNIVERSITY.

1894–95.

CARROL P. H. BASSETT, C.E., Ph.D.

Water Supply from Gravel Deposits . . . . Dec. 7, 1894.

GEN. W. P. CRAIGHILL, Chief of Engineers of the U. S. Army, Past Pres. Am.Soc.C.E.

Notes on Professional Experiences in Connection with River Improvements. . . . . Jan. 18, 1895.

JAMES OWEN, M.Am.Soc.C.E.

The Construction and Maintenance of Water Works in Small Towns. Jan. 25, 1895.

PROF. JAMES HOWARD GORE, Ph.D., Prof. of Mathematics, Columbian University, Commissioner General of the United States to the International Exhibition at Antwerp.

The Decimal System of Measures and its History. . . . . Feb. 11, 1895.

C. C. SCHNEIDER, M.Am.Soc.C.E., Chief Engineer of the Pencoyd Iron Works.

Details of Construction of Engineering Structures. . . . . Feb. 15, 1895.

GEO. W. RAFTER, M.Am.Soc.C.E.

Notes on the Design and Construction of Masonry Dams.. . . . . March 9, 1895.

GEN. FRANCIS A. WALKER, President of the Massachusetts Institute of Technology.

The Restriction of Immigration . . . . April 12, 1895.

ISHAM RANDOLPH, Chief Engineer, Sanitary District of Chicago, Past Pres. Western Soc. C. E.

The Sanitary District of Chicago. . . . . April 19, 1895.

ROBERT E. McMATH, M.Am.Soc.C.E., President of the Board of Public Improvements, St. Louis, Mo.

The City and the Engineer. . . . . April 26, 1895.

The Decimal System of Measures and its History.

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As we make use of a fact, endorse a creed, or utilize an invention, we frequently lose sight of the mental effort which the devising and elaboration cost. We accept them as a portion of our natural heritage from those who have gone before with occasionally the grumble that comes from ungrateful recipients.

In order to rightly appreciate what our ancestors have done for us, it is well now and then to make a halt in our rapid march long enough to contemplate the rise and development of those institutions which are now an integral part of our life, which in theirs had only the beginnings. In such a review we would call up names long forgotten, and realize anew who it was who first placed the principal from which we are incessantly and unconsciously drawing the interest.

Just now when many wise men are glorifying the French Nation for having given us such an elaborate, harmonious and unique system of measures; when learned bodies throughout the land are urging its acceptance; when governments are legalizing or enforcing its adoption, should not some mention be made of that modest priest who suggested that system long before its Academician propounders knew aught of life? Judging from the life of Gabriel Mouton we might infer that he would care but little to have the memory of his notable labors revived, but it is due us to know who it is that deserves our gratitude, and in rescuing his name from practical oblivion others may be led to feel that if forgetfulness is to be their reward some one will find it his pleasant duty to throw aside the surrounding pall and give to their names a newness of life.

In order to fully realize what our hero proposed and accomplished, it is necessary to go back to the beginning of the sixteenth century and

from that vantage ground, view the status of knowledge. At that time the sciences were only on the brink of being, for it would be inaccurate to give the name of science to that mass of hypothetical speculation of which all natural philosophy previously consisted. The purpose of the ancients was to divine natural causes, not to investigate them. The art of examining nature in order to constrain her to reveal her secrets was unknown,—it remained for Galileo to make this discovery. He showed that the human mind is too feeble and too evanescent to progress by virtue of its own strength through the labyrinth of natural facts; that it is necessary at every step to classify those facts and phenomena which approximate to one another. To this, Bacon added the dictum that in the multiplied opportunities which nature offers for inquiry, experiments industriously prosecuted are necessary to conduct to a course of new phenomena which shall neither entangle nor mislead.

It was while the sciences were in this formative period that Ferrara, a city of Italy, gave to the world one who was soon to become famous, John Baptist Riccioli. At the age of sixteen he was admitted into the society of Jesuits, and before he had completed his course of study many regarded with amazement the progress which he made. Rhetoric, poetry, philosophy, and scholastic divinity were his favorite subjects, and these he was called upon to teach in the Jesuit colleges at Parma and Bologna. While engaged in teaching he became interested in geography and astronomy. These topics he found so fascinating and so promising of rich fruit that he obtained permission from his superiors to lay aside all other pursuits that he might devote himself exclusively to those sciences.

While studying everything touching upon his favorite theme that fell in his way he met with Eratosthenes Batavorum by Snellius, a geometer of Holland. In this book was described the means by which Snellius in 1615 had determined the length of a degree of the earth's meridian, and hence, as he thought, the circumference of the earth. This important problem, to know the size and shape of the earth, had received the attention of Greek and Egyptian philosophers and called puerile and ineffectual methods from Fernel, the court physician to Henry II of France and from the Arabian Caliph Almamon.

I may recall to your minds that in order to ascertain the length of a degree, it is necessary to know the exact latitude of two points on the earth's surface. The difference in these latitudes will give the amplitude of the arc which connects the parallels of these points. Now, if the linear distance between these parallels be also known, the length of

one degree will bear the same ratio to the length of this arc that one degree bears to the amplitude of this arc. Therefore this problem is made up of two parts: to determine the amplitude of an arc, and to know its length.

It is said that Ptolemy pointed out the fact that in order to determine the length of an arc it was not necessary to measure along a meridian. Still, no one was willing to accept this statement as true until Snellius demonstrated it to be a fact. He went still further, saying that since it is not absolutely essential to measure on a meridian, it is not essential that the terminal points should be connected by a straight line, but the line may be broken—that is made up of a number of straight lines joined end to end. This, perhaps suggested at once that at least some of the lines might have their lengths computed, thereby saving the trouble of measuring them. It was, of course, known at that time that in a triangle, if one side and the angles be given the remaining sides can be found. The known side might be short, while the computed sides comparatively longer. From this it was but a step to realize that a side which had been computed in one triangle might become the known side of an adjoining triangle, and aid in determining the sides of the latter. Thus triangle could be joined to triangle, link by link, forming what is now called a chain or net, with only one side the *base*, determined by direct measurement.

Snellius measured on the frozen meadows of Sverterwoude a base, and attached to it a chain of triangles stretching from Alkmaar to Bergen. He determined the angles by measurement and the sides by computation step by step from the base. Then, knowing the length of each side, the summation of a set of contiguous lines gave the length of the broken line joining the terminal points. However, these lines not having the same direction, it was necessary to calculate what the length of each line would be if it had the direction of the imaginary line which connected the two ends, that is, he found the projection of each line upon this direction, then the sum of these projections gave the distance required. With this oblique line and its bearing, it was easy to find its projection on the meridian, or the length of the arc which united the parallels of the terminal stations. The determination of the latitudes gave the amplitude and the length of one degree found as has been intimated. This method, here followed for the first time, contains the fundamental principles of all subsequent geodetic operations.

It was the account of these operations which fell into the hands of Riccioli, just at the time when he was anxious to make some valuable

contribution to the world's stock of knowledge. He had already projected a great work, the *Almagestum Novum*, following in the main the plan of Ptolemy's monumental work, but he wished to produce something more than a mere compilation—he desired to hand down some original observations, and thereby stimulate others to reap as well as to gather.

With his mind glowing with such noble aspirations, ready to enter upon any investigation which had the promise of results worthy of the effort, we find him upon Mount Serra-Paderno, whither he had been driven by the heat which had made his winter home intolerable. While there he wrote at least portions of his "Astronomia Reformata" and Chronologia Reformata. In the former he gave the diverse and divergent views of astronomers of all ages, and sought by diligent comparisons and recomputations to bring order out of these chaotic beliefs and deduce principles broad enough to accord with observed phenomena. As one of the divisions of his subject included a discussion of solar units, he found a variety of values for the distance from the sun to the earth and the latter's size. Then, as he looked away toward the Ghirlandina, that graceful tower of Modena's capital, he thought of what Snellius had done in far off Holland. Is it not natural that the thought should come to him that perhaps here was the desired opportunity to enrich the world's knowledge? Perhaps his value of a degree might emphasize that found by his predecessor, and so one more authenticated result could take its place in the "Reformata."

In 1645, in connection with Grimaldi, also a member of the society of Jesuits, he began the task of measuring the arc between Mount Serra-Paderno and Ghirlandina, following in many respects the method devised by Snellius, using a measured base and joining the terminal points by means of a net of triangles. The amplitude of the arc was only one-fourth of a degree, which would not establish great confidence in his results; however adverse criticism is forestalled by our ignorance as to the length of the standard which was employed.

But, as it frequently happens, the work of Riccioli in this direction cannot be estimated by the value of its immediate results. As the Chinese pay homage to the parents of a great man rather than to his children, so should we honor Riccioli who laid the foundations for him who proposed a decimal metrology, and not to the French nation which merely adopted it.

In the mutual interchange of courtesies a copy of Riccioli's work found its way to the College of St. Paul at Lyons, where it met at the hands of Gabriel Mouton a cordial reception. Of Mouton unfortu-

nately but little is known. We hear of him as choir-master at the collegiate church, and can find him only rarely mentioned. He had sent a copy of tables of some trigonometric functions to the Academy of Sciences at Paris, and the Secretary, in presenting it to the Academy referred to Mouton as "*un mathematicien tres habile,*" but this characterization was a recognized formula of the polite Secretary. The encyclopaedias have but little to say about him, and no bibliography gives more than the title of the one book which he wrote. The title of this book has an unattractive beginning: "*Observationes diametrorum solis et lunae apparentium,* . . . " nor is anything startling or of importance mentioned until after fifty-one words are given. One rarely reads a Latin title so far as this, and thus it is that so few have noticed "*Huic adjecta est . . . una cum nova mensnrarum geometricarum idea,*" and even if one should read all the way through it is not probable that any great expectations would be aroused by this "*nova idea.*" In recent times, at least, but few persons have had the opportunity of even seeing the book, judging from its apparent rarity, as evidenced in a two years' search through the libraries of this country and Europe, a search which revealed the existence of only one copy—and that one is in the Boston Public Library. And several standing orders with the book-dealers of Europe and America has not brought to light another copy. This scarcity does not detract from the credit due the expounder of this "idea," but merely explains why he has rested so long in this comparative oblivion.

Let us now see what this scheme is which occupies pages 427–448 of the above named work. He begins very modestly, thinking perhaps others may have hit upon the same or similar system, nor does he desire to lay any claim to priority. He says: "Although here matters of measurement are discussed, they are perhaps not new to all, from the fact that in the vast variety of measures that are in use to-day in all parts of the world it is possible that these which I propose may coincide with some of them. Nevertheless these measures are new both as to their nature and the method by which they are determined and protected against any danger of alteration, and also in the fact that each single measure is composed of ten of the next lower denomination and is itself the tenth part of the next higher—a relationship which is the most important in geometry, affording in geographical calculations great facility with brevity and certainty."

The English may try to claim for Thomas Williams the honor for having suggested the general idea of a decimal system, but his book entitled, "Method for Fixing an Universal Standard for Weights and

Measures," was not published until 1788. The hero worshipers of France seek to find in the writings of the Cassinis a hint of such a system, but Jacques Cassini, the first of that illustrious family who concerned himself with geodesy, was not born until 1678, while Mouton's book was published in 1670. Just when this scheme was first proposed is not known, but it was prior to 1665, as some of the observations in connection with the fixing of the standards were made on the 8th of March of that year.

In addition to having proposed a decimal scale of notation, which in itself is worthy of our commendation, he went far beyond the expectations of his times, by taking the length of his unit from the length of a terrestrial degree, using one minute of this degree for his longest unit the *milliar*. Going down the scale from *milliar* we come to *centuria*, meaning by hundred; then *decuria*, by tens; and *virga*, which was to be the fundamental unit. To know exactly how far to carry this subdivision of units it was necessary to know the length of the degree of the great circle of the earth, for the earth was then regarded as a perfect sphere. Just here we see the influence of our Jesuit astronomer. Mouton says: "Of all the observations that I know, ancient as well as modern, those of John Baptist Riccioli please me most, both on account of their wonderful harmony and their singular diligence which the above mentioned author has exhibited in treating them, and also the industry manifested in the labor which he bore with an unwearied mind for the sake of the truth which was to be attained. Indeed, I have such confidence in these observations that I should regard my own, if I had any, as inferior to them. But, hitherto, I have been unable to accomplish anything in this subject although I am very fond of such things."

The industry and labor referred to were expended by Riccioli in collating and comparing ancient earth measurements to see how they would harmonize with the work of Snellius or his own. He saw in this comparison an endorsement of his value for a degree, that is 64,363 Bolognia rods. Mouton accepted this value, and taking one-sixtieth of 321,815 feet, to which the above is equivalent, he gave 5,363.58 feet as the length of his *milliar*, and one-thousandth of this, or 5.363, 58 feet for the *virga*. With wonderful acumen he perceived that this would be too long for the small unit, which would be in frequent demand, and therefore, proposed a second unit, one tenth as long as the *virga*, which he called *virgula*, saying "the *virga* is the smallest among the large measures, and the *virgula* is the largest among the small measures." Then realizing that still smaller units might be preferable to fractional

parts of a larger one, he subdivided the *virgula* by the scale of ten, giving units which he named, *decima*, a tenth part; *centesima*, a hundredth part; and *millesima*, a thousandth part.

This system can be expressed in English equivalents and is, if we take the most authentic value for the length of a mean degree, as follows:

milliar,	72908	inches.
centuria,	7290.8	"
decuria,	729.08	"
virga,	72.908	"
virgula,	7.2908	"
decima,	.72908	"
centesima,	.072908	"
millesima,	.0072908	"

In order to aid in the introduction of his system Mouton ascertained the relation between the foot of Bologna and that of Paris, perhaps hoping that the French Government might see its merits and authorize its adoption; or he might have desired that it receive the commendation of the Academy. To give the reader an idea as to the length of the virgula, he had it, with its decimae, and one decima divided into centesimae, printed on the margin of a page, with the naïve acknowledgement that it is not given with such accuracy that copies could be taken from it as if it were a standard, a remark emphasized by the careless trimming of the binder.

In 1673 Huyghens published his famous treatise concerning the movements of a pendulum, wishing simply to establish his claim to priority in the application of the pendulum to clocks, but he had announced his invention to some friends as early as 1658, and had made some clocks about this time. Galileo had already shown that pendulums freely suspended were isochronal, and others had proven that the squares of the number of vibrations of two pendulums are to each other in the reciprocal ratio of their lengths; but it remained for the transcendent genius of Mouton to combine these principles and establish the fact that the length of a pendulum in terms of any given measure which makes an oscillation in a fixed time can be made to preserve that measure and re-establish it should the standard become damaged or lost—a method which was subsequently adopted by the English Parliament without any reference to Mouton its inventor. He, however, gives due credit to Huyghens when he writes: "In making use of the following and similar experiments a very exact knowledge is required of the time that has elapsed in the meanwhile. In order to obtain this knowledge

we must have recourse to the clocks of Christian Huyghens, clocks which are constructed with hanging weights.

This Huyghens was a man of remarkable learning, and one to whom posterity will always be indebted for his great assistance in mathematics. His clocks excel all others, and correspond so nearly to the daily revolution of the sun that nothing more accurate can be hoped for."

Thus with a clock, which he regulated so as to record exactly twenty-four hours while the star Serius was making a complete apparent revolution, he made a number of experiments which resulted in his knowing the length of a pendulum in terms of the virgula, that made an oscillation in a second of time as indicated by this clock. Nor did he stop here. He was so convinced of the accuracy of his work and the correctness of his method that he suggested that all nations could determine the relation of their standards to his by simply ascertaining the length of the seconds pendulum in terms of their units—that is to use the pendulum as a go-between. By way of illustrating this procedure, he gave an example, stating the observed data with the principle or rule involved and working out in detail the numerical calculation.

If we should compare this system with the metric measures of length—to be discussed later—we should find at least two important points of superiority: the unit is derived from one minute, and is therefore an exact part of a degree, quadrant and circumference, while the metre has nine degrees as its smallest multiple; then the names are etymologically far better—centuria, by hundreds is preferable to hektometre, and all the terms are from one language, not from two—Greek in addition to Latin. Then again those who use the metric system find that the metre is rather long for the class of measures which we express as fractional parts of a foot, while it is too much of a jump to go to the centimetre and very few use the decimetre. The virga, about six feet, would come in very well for expressing distances for which we now use yards. If these views be correct, Mouton's duplex units, virga and virgula, are preferable to the metre.

It is hoped that what has been said establishes the fact that Mouton was the inventor of the decimal system of measures and the originator of the method of preserving linear units by means of the pendulum. The effort has also been made to show that he acknowledged the assistance which he received from Riccioli, how Riccioli did not forget to mention Snellius and the latter in the very name of his book erected a monument to Eratosthenes, the first to attempt to measure the length of a degree. But have Mouton's successors shown equal magnanimity?

If so, would his name have been so long forgotten and his book preserved by accident only?

When Louis XIV expressed the desire that his reign be rendered still more illustrious by having it witness operations looking towards the determination of the size of the earth, and asked the Academy, his scientific council, to nominate a man to place in charge, Jean Picard was selected. This was a natural choice, because in founding the Academy, he was the representative of the astronomic side.

At this time the earth was regarded as perfectly round, so when Picard undertook to obey his instructions he deemed it necessary to determine the length of one degree only, since all degrees must be of the same length. Following the plan of Snellius, he measured a base and developed upon it a chain of triangles, using for the first time a telescope attached to a graduated circle for measuring angles. Picard, divining that his work was to bear fruit of a lasting character, desired that his unit of length be more securely fixed than in a rod subjected to careless handling or even loss. He therefore constructed a pendulum making an oscillation in just one second, and ascertained its length in terms of the toise which he used, and called it the Rayon Astronomique. One fourth of this length was to be the universal foot, and its double was to be the universal toise. He cautiously suggested that these universal measures pre-suppose that the change of localities causes no change iu the length of the seconds pendulum. He knew of Richer's experience with the clock at the Island of Cayenne, and also said that observations made at London, Bologna, and Lyons seem to show that the pendulum beating seconds must be shortened in approaching the equator; however he did not regard these observations as conclusive especially since he added that the seconds pendulum at the Hague is the same as the similar pendulum at Paris.

At Lyons he met Mouton, this we know, for he says in discussing some observations made there: "M. Mouton in his discussion of a universal measure, stated that at Lyons a simple pendulum whose length was a Paris foot—a length given him by Auzout—made 2,140.4 vibrations in half an hour, from which he concluded that the length of a seconds pendulum would be 36 inches and 6.3 lines."

It is impossible to tell whether Picard borrowed his idea of the application of the pendulum to the preservation of linear standards from Mouton or made the independent discovery, but we do know that he takes Mouton's words when he says: "If one had the length of a seconds pendulum expressed in the usual measures of each country, one could know the relations of these measures as if they had been

compared with one and another directly, besides this, one could detect at any time in the future a change in their lengths."

In the light of modern criticism, Picard's result reveal a unique compensation of errors. He evidently used a false toise, as is seen from the ratio which he gives for his universal toise, deducible from the seconds pendulum, to the toise of Paris and his latitude determinations were uncorrected for aberration and nutation. These errors, strange to say, were so nearly eliminated by mistakes of computation that a subsequent discussion of the whole work gave for the length of a degree a value only 14 toises greater. This work was lifted into great importance and received the stamp of the world's approval because it gave to Newton a value for the earth's radius which enabled him to establish the hypothesis of universal gravitation.

Newton had attempted to prove the theory of universal gravitation by comparing the force of gravity on a body at the moon's distance with the power required to hold the moon in her orbit. He used in his computations the diameter of the earth as somewhat less than 7,000 miles. The result failed to show the analogy he had conceived, but twenty years later, when Picard's length of a degree was made known, increasing the diameter of the earth by about 1,000 miles, Newton was able to demonstrate that the deflection of the orbit of the moon from a straight line was equal to a fall of sixteen feet in one minute, the same distance through which a body falls in one second at the surface of the earth. The distance fallen being as the square of the time, it followed that the force of gravity at the surface of the earth is 3,600 times as great as the force which holds the moon in her orbit. This number is the square of sixty, which therefore expresses the number of times the moon is more distant from the centre of the earth than we are; this required a diameter of 8,000 miles for the earth.

Newton recognized the force drawing an apple to the ground from a tree-top, a stone from the loftiest structure, a drop of water from the highest cloud to be the same as that which draws the moon to the earth, both to the sun with an equalizing centrifugal force to keep each in its place. But he did not regard an hypothesis as sufficient; it needed verification, so when at twenty-three from inaccurate data his demonstration failed, he laid aside this theory, so brilliant in conception, so insufficient in action. Had Picard announced his results fifty years later, the ripeness of the time would have passed by with only Newton's failure to check the search for that grand essential theory without which we could have no exact astronomy, no celestial mechanics. The French geometer stumbled more wisely than he knew, the English

philosopher harmonized theory with fact, applied the finite to the infinite and harnessed the worlds with invisible traces.

The ready acceptance of this theory of Newton, and its demonstration resting on the results of Picard emphasized the accuracy of the latter. Thus it was that when the scientific world was divided into two hostile camps concerning the figure of the earth—the one believing in the prolate earth while the other could see only an oblate form for it—mathematics had advanced to such a point that it was evident to all that the question at issue could be finally settled by determining the length of two or more degrees of the meridian under different latitudes. If the northern degree was longer the earth would of course have the oblate form, if shorter the form would be prolate. The French savants saw in Picard's short arc a beginning for this crucial work, and means were soon obtained for its extension.

Colbert, the liberal minister of Louis XIV looked with favor upon this scheme and placed the work in the hands of Cassini. He proposed an arc of about eight degrees, whose axis should be the meridian of the Paris Observatory. Accordingly he sent his associate, De la Hire, to begin work at the northern end of the arc, while he made a commencement at the southern end. But operations were soon interrupted by the death of Colbert, and the wars which followed prevented a resumption of the work until the year 1700. At this time the king's attention was called to the importance of the undertaking and operations were, upon his order, resumed.

The work began at the southern extremity of Picard's arc and was carried forward until the southern boundary of France was reached. The reductions seemed to show that the length of degrees increased towards the equator, agreeable to the theory of Cassini as announced already by him. However the increase was so slight and the weakness of the work so apparent that it was thought best to measure degrees widely separated in order to definitely settle the question of the increase of degrees, so Cassini, under royal instructions, decided to continue the are northward, and regarding Picard's arc as beyond reproach, he affxed his triangulation to it. In a short time the arc was extended from Dunkirk to Perpignam.

Thus it was that this long arc was at least a reconnoissance for the subsequent operations which had as their moving cause the desire to secure a fixed and definite basis for a standard unit of length.

Picard, after his celebrated arc measurement, devoted considerable time to the determination of the length of the pendulum beating seconds, and had visited several places in France for this purpose. As

already stated one of these places was Lyons, where he met Mouton. So when he returned to Paris he was full of the idea of a universal measure, and when proposed in 1790, the time was ripe for a change. The revolutionary spirit was so rife that there were no sentimental associations connected with old things to interfere with the introduction of new ones. In April, 1790, when the National Assembly first felt the fullness of its power, in spite of the anxiety caused by the news of the rupture with Spain and England, notwithstanding the disorder and trouble in the provinces and the resistance to the decree selling the property held by the churches, and in the face of the many complications which surrounded the reorganization of the judiciary it was ready to take up the question of a new system of weights and measures referred to it by the third estate.

This important matter was referred to M. de Talleyrand, the minister of foreign affairs of the new regime. In his report he said: "The great variety of our weights and measures and their irregular multiples of one another cause considerable confusion of ideas and are the source of embarrassment to commerce; besides, those now employed are not only subject to error but different lengths have the same name. Such a motley of units is a great snare for trusting persons—a species of deception that is more widely practiced than one would imagine, since for each name to which usage seems to have attached a fixed length there are a confusion of ideas and multiplicity of lengths. Nothing can justify such an abuse—nothing but the National Assembly can rectify it."

Then after reciting with that clearness which characterized the utterances of this scholarly minister, the many advantages which would attend the adoption of a uniform system of measures with a unit taken from nature and invariable as nature herself, he proposed that England be invited to coöperate with France in fixing upon such a standard.

"Each of the two nations," he added "should construct for itself a standard bar representing this unit and preserve it with the greatest possible care, so that if centuries after one should suspect a change in the length of the siderial day these bars could be called on to definitely decide the matter. Perhaps one can see in this united interrogating of nature for the purpose of establishing such a standard the beginnings of a political union brought about by the intervention of the sciences."

This proposition of Talleyrand was referred to a commission, which through its secretary, M. de Bonnay, submitted a report on the 8th of May. The commission, after having recounted the advantages which would result from the adoption of a universal system of measures, said that England was ready to join in fixing such a system and added:

"When these two nations, which have only one another as rivals, adopt it, all of Europe will follow." It then submitted a series of resolutions providing for the collection of all the fundamental units in use throughout the provinces of France, their comparison with the new unit when obtained and a formal invitation that England authorize the Royal Society of London to appoint a commission to coöperate with a commission equal in size and appointed by the Academy of Sciences of Paris in determining wherever mutually agreeable the length of the seconds pendulum on the forty-fifth parallel—or some other parallel—and to deduce from it a universal unit of length.

From this it will be seen that the first universal unit proposed was one derived from the pendulum—the original plan of Mouton.

One would thing that a project so warmly endorsed by every one would find its immediate adoption, but an unexpected delay came in the shape of a suggestion that if uniformity is vital for a system of measures it would be even more important for the monetary system. Hence the proposition of M. Bureau de Puzy to incorporate a decree fixing a new denomination for the currency delayed the signing of the two-fold decree until August 22.

However, the National Assembly was not losing sight of this universal unit, it was strengthening itself along all lines and with this in view communicated its plans and purposes to the Academy of Sciences with a request for suggestions. The Academy took its time to consider the question for it was on the 26th of March, 1791, that M. Talleyrand ascended the tribune, and putting an end to the rather stormy discussion then in progress on the subject of frontier defense, read the communication from the Academy, signed by Condorcet as secretary.

This wise body was not at all certain that the seconds pendulum was the most reliable standard but suggested that it was more important to adopt as a unit a length that could be re-established with absolute exactness than one that merely admitted of easy recovery. They also showed their wisdom in saying that they, in defining such a standard, would be working not only for France but for the world, and for all time.

To this communication M. Talleyrand added: "You know that the units which might be employed are reduced to three, the pendulum, a quadrant of the equator and a quadrant of the meridian. After a serious consideration the Academy has decided to recommend the last named, its committee merits your entire confidence and the whole question ought to be definitely settled. The substance of the resolution which I now submit is the joint labor of M. M. Lagrange, Lalande, Borda, Laplace, Monge and Condorcet."

These resolutions were: "The National Assembly considering that in order to establish the uniformity of weights and measures, it is necessary to fix a unit both natural and invariable, and that the best means of inducing other nations to adopt this system would be to choose as the basis something that is not peculiar to one country alone but common to the entire globe, therefore in view of the opinion expressed by the Academy, be it resolved: that it (the Assembly) adopt the length of a meridional quadrant of the earth as the base of the new system of measures, that agreeable to the Academy's suggestion the measurement of the arc from Dunkirk to Barcelona be immediately executed, that the king charge the Academy to appoint a commission to carry on these operations in concert with Spain whenever Spanish territory is occupied." These resolutions also provided that the unit, to be called the metre should be the one-ten millionth of this quadrant. The decree was signed by Louis XVI, on March 30, 1791.

The commission was at once appointed and Delambre and Mechain sent into the field to begin operations. It is interesting to note that on June 10, 1792 an edict was issued to the authorities of all the departments through which the arc extended to assist the observers whenever possible, to furnish them with transportation and to protect their signals. This edict was signed only two days before the downfall of the Girondist party, at a time when one would ascribe to the leaders thoughts of a more personal character.

Unfortunately this command soon became a ground for persecution, and those to whom it was shown considered it their duty to throw obstacles in the way of its holder. It is doubtful if men ever worked under greater disadvantage than those which surrounded Delambre, he was stopped at every village, his pass ports were scrutinized and in every instance found irregular, his instruments were examined, and as he naïvely remarks, he was repeatedly called upon to give lectures on geodesy to people who could not even count. After a while a republican form of government came into power, and as is sometimes the case a new order of things came into force. A new commission on weights and measures was appointed, the Academy was dissolved and all who held positions of trust under the old regime were regarded as hostile to the new. Delambre was deposed, brought back to Paris and carefully examined regarding his political views and especially as to his hatred of kings. As usual, charges were made that he was not progressing rapidly enough with his work, when it was found that he had been wise enough to change parties as fast as his superiors had. But even this charge was dismissed and he was allowed to resume his work.

Mechain who was wholly outside of French territory fared almost as badly, having to contend with superstitious and ignorant people he was continually hampered and his progress greatly retarded.

The National Assembly felt sure that it could rely upon the coöperation of England, but unfortunately England either for lack of sympathy in the evident ascendency of a republican form of government or not wishing to share the glory of establishing a new unit with another nation, declined to appoint a commission to meet the French commission. This refusal piqued the Assemblymen and in the fullness of their new-born power passed a resolution on August 1st, 1793, in which they denounced to all the world the English people and the conduct of the English government. It also strengthens itself and the doubting spirits of France who were not clamoring quite so loudly for new things by adopting a decree defining what the metre was to be and declaring that its use should be obligatory on July 1st of the following year. This decree contained a table of measures which was magnanimously offered to all the peoples of the world.

As is frequently the case the date set was somewhat anticipatory. The difficulties surrounding the completion were far greater than had been supposed, and even when the observations were finished and the computations ended there were a few jealous persons who had not participated in the work who felt sure for that reason that the work was not reliable. They by dint of well-directed clamors succeeded in having a revisionary committee appointed, and this committee was not satisfied with a mere perfunctory examination. The entire work passed under a very careful scrutiny—from the standardizing of the base bars down to the last astronomic observation.

It should be said that the members of this commission were delegates from various countries sent upon the invitation of the French government. This will explain how it came about that as members of this revisionary committee we find the name of a Swiss, a Hollander and only two Frenchmen—Laplace and Legendre.

In addition to revising the observations and computations they suggested a modified system of units. The original proposition of the Assembly contemplated having milliare as the only multiple of the metre with the subdivisions just as we now have them. It is important to note that this multiple is the same as the one first proposed by Mouton.

But the revisionary commission changed this system so as to give to each multiple by ten a name of its own, that is decametre, hectometre and kilometre. This plan did not meet with universal approval and in

fact was opposed by Delambre as being cumbrous, useless and philologically incorrect.

However in adopting this new scheme the Assembly very wisely annulled the decree which gave a new subdivision to the year and the day.

The new metre was defined as containing 3 feet 11.296 lines, so that in reality the old toise of the Chatelet was the standard, the only advance being in providing for the construction of a platinum bar on which a definite portion was to be marked off—called a metre—and the careful preservation of this bar. The ten years of laborious work, countless hardships and months of weary computations had as its only tangible result the decree that a certain length should be safely kept, and a definite part of it regarded as a unit.

Luckily Borda retained something of the legacy handed down by Mouton and determined the length of a pendulum beating seconds at Paris in terms of this new measure—this value expressed as so many toises of 1668.

We now know that the length of the quadrant is not what this wise commission thought they found it to be, nor are we prepared to say just what their error was. Geodesists have not yet agreed as to the exact shape of the earth, the value for the ellipticity changes with each new calculation, and even the permanency of a meridian section is by no means assured. This unfortunate state of ignorance has a worse result than keeping us from knowing the true length of the metre: it keeps in perpetual doubt the accuracy of every artificial boundary line, no map of large areas can boast of absolute precision at the present time and all measurements resting upon the earth's diameter as a base are uncertain.

It is hoped that success has attended the effort to show that Mouton was the first to suggest a universal system of measures deduced from an earth measurement and recoverable by means of the pendulum, that his work and direct acquaintance influenced Picard in prosecuting observations with a pendulum, observations which were preserved; that Picard's influence gave life and importance to each and every suggestion coming from him, and that this influence rested upon the fact that it was his value for the radius of the earth—a value accidentally somewhat near correct—that enabled Newton to establish the sufficiency of his theory of universal gravitation; that this aid to Newton gave to Picard's work the stamp and appearance of great accuracy, so that when the new government came into power and desired to lose sight of every thing that could call to mind a former regime, and thought it well to begin

with the system of measures, it was Picard's arc that seemed to stand as a potent invitation to take it as a contributing integral part of the quadrant that was to supply the multiple of the would-be standard; that in the first nomenclature one of Mouton's terms was proposed and that as a crowning part of the work of the metric commission the length of the seconds pendulum in terms of the new unit was carefully determined.

If these theses have been sustained, should we not expect some acknowledgement of Mouton's suggestion? some laudatory, or at least commendatory, remark to show the source if not the extent of the indebtedness which the Assembly owed to him?

What did he receive?

On the 17th of June, 1799, a bar bearing upon its surface two marks whose distance apart was one-ten-millionth of the earth's quadrant was formally presented to the two Conseils du Corps Legislatif. It is quite natural that the members of the Commission des poids et mesures should rejoice over the conclusion of their work, a work which had withstood at the time the severest criticism, nor is it surprising that their spokesman should say: "To employ as a fundamental unit of all measure a type taken from nature herself, a type as unalterable as the globe which we inhabit, to propose a metric system all of whose parts are intimately interdependent, and whose multiples and subdivisions follow a natural progression, simple, easy to comprehend and always uniform is certainly an idea beautiful, grand, sublime and worthy of the brilliant century in which we live." In the detailed account which is then given there is interspersed a large amount of praise for the participants—from Talleyrand who first laid the proposition before the Assembly on May the 8th, 1790, down to the laborers who carried in the prototype on this memorable occasion. But one looks in vain for even a mention of the humble, conscientious, credit-giving priest who was the first to propose "a type taken from nature herself, as unalterable as the globe which we inhabit;" in its stead, however, we find a medal bearing the inscription:

"A TOUS LES TEMPS, A TOUS LES PEUPLES."

Notes on the Construction and Maintenance of Water Works in Small Towns.

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The ambition of the young engineer to design and superintend works of large magnitude and thereby gain professional glory is praiseworthy, but in only a few cases is such distinction achieved, consequently the average student must for years to come content himself with the exercise of his talents in what he may consider small problems, yet when it is considered that the aggregate of the world's industry and enterprise is composed of such small items, he will still find himself in the race by careful and conscientious attention to such work as to his lot may fall.

The construction and maintenance of water works is an economic problem of engineering, and one of the few phases of engineering in which by the consensus of general opinion, and with very few exceptions is the matter of dollars and cents rigidly adhered to. I say this premeditatedly, because in many other matters of such municipal development the question of return of money investment is not considered or cared for; but in water works, strange to say, most municipalities seem to strain a point, even if they own the plants, so that the burden of the maintenance may not be taxed on the community at large, while in cases where a private corporation owns the works the only question to be considered for the company is to make it pay.

It will be seen, therefore, at the outset of any proposed water works construction the financial question is paramount in controlling the engineer as to the general design of his works at the present and its future development hereafter. It may be readily seen that a town may be so situated both as to elevation and remoteness of available water supply, as to render the proper construction of the means of such supply far beyond its capacity to render a renumerative return for the invest-

ment. It therefore follows that such a town must sacrifice some other important want or desire to enable it to have water at all.

In another case the facility for the introduction of water may be so great and the cost of administration so low as practically to enable the municipality, if it owns the works, to luxurious development or the expenditure of the surplus for another object and in the case of a private company to very satisfactory dividends.

At this point I might allude to the engineering status of water works construction in its relation to the clientage whether public or private. A private company organized as a money-making institution as a rule demands and gets the best talent in its special line it knows of, and as a rule is willing to pay for it. The municipal corporation on the other hand is very apt to tack on the work of construction of its water works to the already over worked and underpaid City Engineer, demanding of him not only a large amount of work but very probably to him new lines of investigation and study, and if he is up to the standard he gets along, but if the work is beyond him, failure is inevitable and an unwarrantable increase of money expenditure is the consequence; and such failure and such extra expense is thrown on the shoulders of the Engineer, whose career may thereby be unwittingly marred for all time.

I therefore repeat that in water work construction above all, the Engineer will find his hands tied, his plans criticized, and every possible rule of engineering practice hampered by the plain bold statement of *Will it pay?*

The Engineer then in considering the problem of constructing a water supply for any town will find the following problems facing him:

1. Money, either as to the capacity of a municipality to raise it or the willingness of a company to invest it.
2. The demands of consumption.
3. The possibilities of supply.
4. The characters of such supply.
5. Ease or difficulty of its introduction.
6. The problem of distribution (static).
7. The method of distribution (dynamic).
8. The economy of maintenance.

I wish here to make the statement that I am not delivering an essay on water works construction, in which is declared all the general principles and calculations entering into the work that you can find in the many books written, nor do I propose to allude to the different formulae extant on the flow of water through pipes. All that you can undoubtedly find in the libraries at hand, and it should be part of your

duty to absorb all and everything available, provided, what you have is stamped with authenticity and not marred with error. My idea is to give you such points as are arrived at in practice and become, if I may say, the traditions of the profession, and they are points which may or may not be alluded to in the books at hand. I came across the other day in a book detailing how to wipe a joint, which was very interesting, yet I am satisfied that a week's practice in a shop under the eye and with the points of a practical man, would be far more of use than the book.

Under the first head, that of *Money*, I alluded to in my opening, that in water works owned by the municipalities is usually raised by bonds. The crystallized experience of municipal indebtedness limits its bonding power for all purposes to 10 per cent. of its valuation. If the water works are to be the first issue of bonds, more liberality can be allowed for that work than if, as is often the case, bonds have been issued for street openings, sewers, and municipal buildings, leaving a smaller sum left for the water. Then the question arises, is there money left for a proper plant? In some cases there may not be, and no works can be constructed unless a private company steps in. In case there is margin left, it may be only enough to build a portion of the works of proper design or the alternative is left, and this alternative is one that tries an engineer's soul, of putting in inadequate and cheap construction which in a few years will have to be torn out and replaced when more money is at hand or the community have acquired more knowledge of the necessities of the case. I say to you, however, very positively, that it is a dangerous plan at any time for any engineer to design any work that he *knows* is inadequate to the requirements. Criticism and blame are sure to follow, and perhaps undeserved. If he does not know any better, the blame of course is merited.

For instance, if you have a town of 15,000 people with a valuation of $9,000,000 giving a bonding power of $900,000 ; this town has a debt, say, of $300,000 for streets and $200,000 for other development, leaving $400,000 for the water supply. This amount in a majority of cases is enough to install the plant. It may happen, however, that the conditions demand a larger outlay. Where is the curtailing to be done? If you leave a part of the town out of the service an outcry is made and a veto results. So the pressure is put upon the engineer to economize on his dam or his conduit as being good enough for the present, and the engineer reluctantly consents, knowing that money spent on inadequate construction for supply is in nine cases out of ten wasted when enlargements are made. Yet generally the curtailment is done in

that direction. Whereas, if the amount of pipes had been limited at first and proper work done for the supply, no money would be wasted and those not getting the supply at first would be sure to get it afterwards.

If a private company starts to supply a town, it first examines the chances of supply, the possibilities of consumption, the capacity of the consumers to pay, and the willingness of the town itself to add to the revenue in the shape of hydrant service; and it considers the growing tendency of the municipality with a view of adjudging the paying capacity of the investment both in the present and in the future. It is an interesting fact that a majority of the towns of the United States are supplied by private corporations and the avidity with which their stocks and bonds are absorbed by the public at large is conclusive proof of their paying capabilities.

In some cases, however, the rapid growth of a town necessitates the continued expenditure of money both for supply service and delivery service, debarring an immediate return of the investment or necessitating a constant increase of capital account. Time, however, solves the problem, and generally satisfactory to the investors.

THE DEMANDS OF CONSUMPTION.—The consumption of water is generally rated at so many gallons per head per day and varies from 18 to 180 gallons per head in the varying municipalities.

So large a difference leaves the matter open to the judgment of the engineer and necessitates a close examination into the requirements of the particular community, the business and habits of the people, the variations of temperature, and the pressure of the water delivered.

It may safely be asserted that with ideal conditions both of service and people, no more than 30 gallons per head per day should be used; more than that is waste; less than that may be considered niggardly. These figures being purely ideal no engineer can of course safely adopt them. So the general present practice under general favorable conditions is to allow 60 gallons per head per day. More than this is used in hot and dry weather and also in extreme cold weather. The largest consumption in some places being in the summer, and in others, in northern latitudes, in winter. This standard of 60 gallons would be safe only for a city up to 40,000 people, larger towns consuming the water in larger quantities as the population increases—New York 120 gal., Philadelphia 140 gal., Chicago 140 gal., Milwaukee 160 gal.

It must be remembered, however, that with the problem of municipal economy that must inevitably face the people of this country, the economy of water consumption is one that can be surely counted on,

and in designing works it will be a factor of considerable moment. The increase of population is inevitable, the available increase of water is uncertain and economy must be the result. This economy can be arrived at by the rigid supervision of pipes and plumbing and by the use of meters. Both practices are being systematically used in different places and with very encouraging results.

In one case to my knowledge, the consumption of a large city was reduced from 100 to 60 gallons *per capita* by a careful and detailed inspection of the service pipes, and in a neighboring town was reduced to below 20 gallons by the enforced use of meters. When it is considered that the city of London uses less than 30 gallons per head, and Liverpool less than 60, and many towns in Europe only 25, it may be readily seen that the possibilities of economy in this country have been merely suggested.

One great difficuty to overcome is to divest the public mind of the idea that water should be as free as air, an idea that is curiously very prevalent, and with that idea divested and the further idea inculcated, that water is a commodity to be paid for as used, a great deal of the trouble will be overcome. Many cities have encouraged the idea of free water by furnishing it to manufacturers at very low rates, the cost to these cities of the water itself has been low and they have decided on such a line of action as a good business investment. I therefore repeat that it will be safe to assume for supply of the town the basis of 60 gallons per head per day, and in making calculations for such delivery care should be taken to arrive at a definite idea of the increase of population. This increase can be ascertained by tabulating the census returns for the past thirty years, and on the basis of these returns calculate the probable population forty years hence and make that amount multiplied by 60 the required amount of water. I use forty years as about the limit. In many cases it may be found more economical to only allow for twenty years' increase and let the subsequent period take care of itself; but a result of that kind will depend upon the peculiar local conditions and requirements.

THE POSSIBILITIES AND CHARACTER OF SUPPLY.—Having determined upon the amount of water necessary, the next point is to get it, and such water is generally taken from:

1. Rivers with a dry weather flow much greater than any probable demand.

2. Rivers with a dry weather flow equal to the demand.

3. Rivers with a dry weather flow less than the demand, or with no dry weather flow at all.

4. From surface wells and galleries.
5. From artesian wells.
6. From springs.

In the cases of water supply by river abstraction, the first point to ascertain is the extreme dry weather flow of that river, an average flow is of no benefit. And as a casual measurement of the dry weather flow of any particular year is deceptive, investigation should be made as to the extreme lowness of water within the memory of man, and careful measurements taken to ascertain the exact delivery so that there will be no possibility in the reasonable future of the demand by the people exceeding the supply of the river, as such a condition of circumstances would be a fatal mistake. The mechanical measuring of the flow of water in a river is a matter that can easily be learned; carefulness in manipulation and good judgment are required coupled with some experience.

A straight run of the river should be selected free from eddies and cross currents with as nearly as possible a uniform cross section. The varying velocities from centre to side should be taken and very many timings should be made to arrive at a correct average. My usual practice has been to select a length of 100 feet divided into two 50 foot sections, carefully level the cross sections at each 50 feet, and have ranging sites across the stream. I used a float made in the shape of a tin tube loaded with shot to keep it vertical and with sufficient loading to keep the bottom of the tube clear of the bed of the river. I would time the passage of this float for the two 50 foot lengths taking three timings of each trip and have the float run down the river every five feet in width with many observations on each. The average of the timings multiplied by the cubical contents of the river in the 100 foot section would give the flow of the river, from which 8 to 10 per cent. should be deducted for the side and bottom friction. If it should be found impracticable to use tube floats a very desirable approximation can be arrived at by using surface floats and deducting 20 to 30 per cent. of the results arrived at for the flow of the river.

Having ascertained the extreme dry weather flow, ordinary judgment would dictate as to its availability for the special purposes required. But it must always be remembered that in all cases liberal margins should be allowed, for in all questions of water supply, extremes of dry weather heretofore unheard of may occur and special demands for consumption may be suddenly created; and in such cases this liberality will be appreciated.

The other question entering in with a supply of water from large

rivers is that of sewerage pollution, and that problem is now agitating various cities in different sections of this country.

It would not be within the probabilities that an ordinary intelligent engineer would select a positively polluted stream for the water supply of any town, and I would be loath to say that anything of the kind has been done. Yet the fact is undeniable that many cities are being supplied with water positively and aggressively polluted by the sewerage of the cities on the same river above them, and the difficulty through the large increase of population is daily being aggravated. In my neighborhood, Trenton runs its sewerage into the Delaware river, the waters of which are used to supply the cities of Burlington, Bordentown, and Camden. Philadelphia uses the water of the Schuylkill polluted by the sewerage of Reading, Polestown, Phœnixville, Norristown. Jersey City uses the water of the Passaic river which is used as a sewer by the cities of Paterson and Passaic. And the same trouble is experienced on the Merrimac river by the cities of Lawrence and Lowell. All these waters are being used, polluted as they are, with more or less dissatisfaction and more or less direful results. The doctrine of self purification has been the avowed reliance for the safe use of such waters, but recent bacteriological investigations have thrown great doubt on the purification theory and only the fact of extreme dilution can be given as the cause for the immunity from disease that seems to exist.

As cities grow, their sewerage enlargements throw greater discharges into the outlet and at some time or other an outburst of epidemic will call a halt either to the pollution of the streams, or the use of such polluted water for potable purposes.

The fatal effects of cholera at Hamburg were undoubtedly due to the polluted water used by the inhabitants, and had such an effect that an entirely new source is now in use or about to be used.

Inventive efforts in the construction of filters for cleansing and purifying water are now prevalent with more or less success; but I would say to you, and say it very emphatically, don't select any water for the supply of any town that is polluted or liable to be polluted. Your clients may insist that there is nothing else to do, and in that case I would advise you to let some other fellow do it. Pure water is as much a necessity of life as pure air, and for any engineer to advise the conduction of anything but pure water for consumption by his fellow beings, may not be criminal, but to my mind is something akin to it.

When it is proposed to conduct the water of a stream with a dry weather flow less than the demand, other considerations and problems enter in and the problem of storage is first encountered.

The first thing to do is to find the dry weather flow of the river. In such a case as this it can be done by means of a weir, the drawings and use of which can be found in any treatise on water works and requires no special comment. Having ascertained the flow and knowing your requirements, the amount of storage can be determined upon.

Then ascertain the drainage area of the stream above the proposed site of the dam and if possible collate statistics of rain fall as near the locality as possible. If no records have been kept, a safe basis of 48 inches per annum for all territory east of the 100th meridian may be used, but it is far better to get hold of records.

Then allow one third of the rainfall as the amount of water available for storage. Then design an impounding reservoir for 100 days' supply less the low water flow of the river and the result will be the amount of water available for use for the town. By moving the location of the dam up or down the stream or by making the dam high or lower the requirements of varying localities may be varyingly satisfied provided, that there is area enough and possibilities for enough storage for the population to be supplied. Other and tributary streams may be utilized in the same way connected together, and even streams on another water shed have been dammed up and connected by tunnels with the other supply.

There is also the question of gravity to be considered. It may be found more economical to locate two dams on different forks of the same river and deliver the water by gravity than to pump it with the dam constructed at a lower elevation.

In deciding the location of the dam a supply by gravity or a supply by pumping may at this point be of great moment, and in comparing the two methods the cost of the dam and the length of the pipe on one hand, and the cost of the dam, length of pipe, cost of pumps and necessary structures for their proper use, the capitalization of the cost of running the pumps and keeping them in repair on the other hand have to be considered.

In water plants under municipal control gravity supplies even at a sacrifice of a larger first outlay are to be preferred, as the injection of political influences in the management renders an economical administration difficult to realize except in very special localities.

The cost of a pumping plant has been reduced to a mathematical certainty but the cost of running them varies so with conditions that it is hardly safe to give figures. The price of coal, the price of labor, the amount of water pumped are all varying elements without considering the other question of politics at all.

To show the variation of the cost of pumping I would state that it costs one city $9.31 a million gallons to pump 12,000,000 gallons a day to a height of 140 feet, and a small town adjoining it costs $30 a million gallons to pump 500,000 gallons per day to a height of 380 feet. As these are extreme cases a variation between these two can be considered for any conditions that might arise. But in both cases cited the management is judicious and economical.

In the selection of a water supply by impounding streams the same question of pollution may arise as in the other cases, but not so frequently; and in the selection of any site for a reservoir care must be taken to prevent any such pollution either into the reservoir or into the stream above its source.

The next source of supply to be considered is from surface wells and galleries. I place these two methods in the same category, as the difference is merely one of detail. The amount of water to be collected in any underground can not be determined in any particular location by any rule or calculation, a knowledge of the proper delivery can only be ascertained by previous experience or actual experiment. The geological formation, whether of earth, sand, gravel or rock, is an important factor in the capacity of any territory for water delivery, and it is impossible to give any standard rules for delivery, the variation in the different soils being so great. A 50 ft. well constructed in Prospect Park about 70 feet deep constructed in a sandy soil delivers constantly about 400,000 gallons per day. A well that I constructed 30 feet in diameter and 50 feet deep in sand stone rock delivers 200,000 gallons per day; and these two statements may give you an idea of such delivery.

The construction of galleries, which are merely elongated surface wells, is dependent to an extent on certain defined rules, as they are generally located in valleys parallel to the streams and with a view of intercepting the underground flow which would otherwise flow into the river. Their capacity of delivery is dependent on the friction of the water through the soil which in such cases is mostly sand or gravel; and the ultimate flow is limited by the retardation due to the friction.

Careful investigation has to be made on the liability for underground supplies of this character to become polluted. The character of the soil here becomes of moment and the possibilities of petration and purification of the different materials. Earth through which the water passes with slow velocity and if sufficient distance is allowed will probably absolutely purify water; then sand and gravel to a lesser degree, and rock may be said to have no capacity at all. It must be remembered

that there will be at some time or other a limit to the purification of watér and then either complete stoppage of flow ensues or a polluted supply is inevitably the result.

There are also many supplies of water derived from what is known as driven wells. The same remarks made on open wells or galleries are applicable to them, both as to quantity and quality. They are distinguished from what is known as artesian wells by being of comparatively shallow depth.

The water from artesian wells is obtained from water bearing strata at a greater or less depth, varying from 50 to 2000 feet. The question of the quality and capacity of the water can only be ascertained by absolute test except that general inferences may be drawn by complete knowledge of the geological and topographical formation; but no rules can be laid down.

Supplies from springs are of easy selection. The quantity flowing in dry weather should be ascertained and deductions made as to its capacity for the requirements of the particular case. Both springs and artesian wells are generally free from pollution.

In all these selections of water for water supply the inherent quality of the water is of great importance. Clear water of a small degree of hardness from lime, with a uniform temperature of 50° is the desideratum. All of these requirements, however, are rarely attained. Supplies from the larger western rivers are generally soft but muddy, with a high temperature in summer. Where any stream is used the water is generally pure, outside of the liability to pollution, and soft.

In well water the character varies as to locality. In shallow wells and galleries the water has a moderate degree of hardness with a constant temperature of about 52°. In deep wells there is great liability for an excess of mineral matter, either carbonates or sulphates of lime and other substances which are absolutely deleterious to health.

It is therefore incumbent in the selection of any supply to have the water thoroughly and completely tested both chemically and biologically, as it would be a fatal error to make preparation for a complete system of water works and then find your water bad or unsatisfactory.

The question of temperature I find to be of considerable importance especially with the consumer. Cool water drawn from a faucet even if it has no other qualification is much more popular than a warmer and perhaps better supply.

Having arrived at a general conclusion as to the character, quantity and location of the supply, the next item of investigation is the method of introduction. This may be divided into two general

heads, viz., that of pumping and by gravity. In large river supplies and also in surface and deep well sources the water will invariably have to be pumped and calculations to that end will have to be made. If the supply is to be acquired from an impounding reservoir gravity should be selected by preference if possible, for the reasons alluded to previously.

If pumps are used their location, size, and capacity are dependent on the requirements of the community to be supplied. But it must always be remembered that pumping capacity of both boilers and pumps should be twice the actual greatest consumption, unless ample reservoirs are provided, which is not always economical.

The selection of pumps is not a matter of great concern, as the pump manufacturers of this country have all brought the requirements of mechanism in such art to so uniform a standard, that a selection of names would be invidious and any well known maker of good reputation will give satisfaction.

The specific requirements, however, should be given, based on the gallons of water to be pumped for 24 hours, to the required height, with a boiler pressure of 60 to 80 pounds, with a piston speed of 60 to 80 feet per minute, and with a rating of the boilers per horse power.

In deep wells, however, different contingencies arise. If they are flowing to the surface, or if the water rises to within suction limits, 15 to 18 feet, ordinary pumps can be used. If, however, deep well pumps have to be inserted the delivery of the well is limited to the diameter of bore, and the capacity is based on the pumps and not on the flow of the well. A new arrangement is now being tested whereby compressed air is forced down the well to any desired depth and the water forced up to the surface. Great success has been so far achieved by this method and it bids fair to revolutionize the whole practice of deep well pumping. In one case the capacity of the wells was increased more than four fold.

The next item for consideration is the distribution of the water through the town. In working out a system of piping, the topography of the locality has first to be studied out. If the ground is generally level and it is possible to supply the whole territory by one standard pressure, there is not much complication. First, lay out your best location for your main and if the territory is too large it may be advisable to have two mains, but generally one main should be sufficient. This, as a rule, should not be less than twelve inches in diameter for a town of, say 10,000 present, 20,000 future population. This must be increased to sixteen or twenty inches for larger towns. Subsidary

mains of eight inches should diverge at reasonable intervals. The cross streets should be laid with six inch pipes and in a few isolated cases with a high standard of pressure four inch pipes may be used. It may be, however, laid down as a good rule that no pipes less than six inches in diameter be laid at all. The friction in small pipes mounts up very fast and renders the free pressure uncertain and useless and very likely limits the household consumption to a niggardly basis. I know of a town of 8,000 inhabitants in which only six inch and four inch pipes were laid with no mains at all, the piping amounting to a mileage of 24 miles, where the household service at the high levels is intermittent and water can only be drawn from the mains at night, and the pressure at the hydrants is about 5 pounds. And this result is entirely due to the smallness of the pipes, for the pumping power is ample.

The extremes of elevation in any town with one pressure system should be limited to 75 feet; a greater distance than that necessitates different services at different elevations.

In laying out a system where there is a great difference of level, say from 75 feet to 500 feet in elevation, the plan of piping will have to be arranged to accommodate the different districts which are at different elevations. If the service is by gravity and the whole town can be thus supplied, it will be necessary to regulate the pressure for the different districts by pressure regulators. If the elevation of the dam is only sufficient for the middle and lower service, an auxiliary pumping plant may be put in for the high service; but it is doubtful if it would be economical to use gravity alone for the low service and then pump the middle and high service, unless the low service has from two-thirds to three-quarters of the consumers and it probably then is better to put in a complete pumping plant.

If the whole supply is pumped for such a town as suggested above with a population of 10,000 people, it may be found cheaper to pump the whole service to the high elevation and reduce the pressure for the two lower districts, provided the pump house is at such a distance from the center of consumption as to make the cost of an extra main more than the capitalization of the extra cost of pumping to the higher elevation.

I have a case of my own, where all the water for years has been pumped to an elevation of 375 feet, and the pressure subsequently reduced for the different districts. It was cheaper to do this than to lay an extra main for a distance of a mile and a half. I have now been lately experimenting on the problem of pumping the high service through the main at a pressure of 167 lbs., and supplementing this by a

service from another pump in the same house at a pressure of 80 lbs. through a local service pipe. This is supplementary and merely supplies the ordinary draft. If, however, a fire should occur or the six inch service pipe be inadequate, the regulator between the middle and high service comes into play and the deficiency is immediately made up.

In arranging for the standard pressure service in any town different practices and different ideas are so much in vogue as to render the statement of any standard open to criticism. An extra pressure over 30 lbs. in an ordinary town for domestic service is superfluous, and many towns and cities, especially those owning the plant, do not pretend to give that. Of course satisfactory fire service cannot be obtained with that pressure and engines have to be used. Where fire pressure is demanded, 70 lbs. is a safe standard though many have less and some have more; but make your standard about 70 lbs. and you will, as a rule, give very satisfactory service. It is curious to note that both the domestic and fire service given by private corporations is almost uniformly better than that given by public municipalities, from the fact that the public are much more exacting from an outside corporation than they are from themselves as part owners of the municipality.

In arranging your piping eliminate as much as possible all dead ends and make your circulation as thorough as possible. Divide your piping lengths in sections of not less than 1,000 feet between your gates, and if you can, put in a gate on each hydrant branch. This practice will obviate the necessity of depriving long stretches of houses from water when a break occurs, or a hydrant has to be repaired. My plan now is, to put in four gates at a four way intersection so as to carry the water around any section of street where repairs have to be made. The general practice is to locate hydrants for fire service about 500 feet apart, nearer together in closely built up sections and occasionally to alternate them on each side of the street, as a fire close to a hydrant may render that particular hydrant useless for the time. Having planned out your supply, laid out your system of piping, the next thing to do is to construct your works, and in that there may be found a few items of interest.

First, as to your pipes. Almost universally nowadays, cast iron pipes are used for water distribution. Spasmodic attempts have been made with other material, such as cement lined sheet iron, steel, and wood. I came across a town last year in New York State, near Albany, where wood service water pipes were being laid; but as a rule cast iron is used, and I would advise you under our present enlightenments not to experiment on any untried methods. Of course, as time goes on other materials

may be found. As the cast iron pipe industry necessitates large capital, and so is in few hands, and the material delivered is generally of good quality, and all that an engineer has to do is to limit his weight per length for each diameter and use the sound test for crack and flaws. Most defects can by this means be detected, except unevenness of metal due to the movement of the core, and inside fractures due to careless hammering the sand away after the casting. Of course, there is always a liability for any pipe at any time to break, and it may be years after a pipe is inserted before the break manifests itself. But careful selection of the manufacturer and careful testing as I suggest, will give ordinarily satisfactory results.

The depth that pipe should be laid is a matter open to discussion, and is really dependent on the flow in the pipe, and the locality in which the pipes are laid. In average cities a depth of four feet is ample, provided that care be taken that the house service pipes in crossing the gutters is kept to the same depth, as more often the small pipes freeze than the large ones. In Colorado, I understand, the pipes are laid seven feet deep, and I myself have laid many miles five feet deep. In southern cities of course no such depth is required where frost is never encountered, and so has not to be taken into consideration.

In laying the pipes care should be taken that the ground is trimmed truly even where the pipes rest, for if a hump is left in the middle, the filling of the trench afterwards is liable to crack the pipe. A chamber should be dug for running and caulking the joint and water kept away while the lead is being poured. Skill and care are required in caulking a joint, and if a contract for laying water pipe is made, it is a good idea for the company to hire and pay the caulkers themselves.

The amount of lead required for a joint in a 4 inch pipe is 4½ lbs.; 6 inch pipe is 7 lbs.; 8 inch pipe is 11 lbs.; 12 inch pipe is 14 lbs.; 16 inch pipe is 15 lbs.

In refilling trenches care should be taken that no large stone fall on the pipes and crack them.

There are one or two points of interest in the construction of dams and impounding reservoirs, although these matters are so fully treated by engineers that what I may say may be considered nothing but repetition.

Having selected the site of the dam it is better to clean off all top soil, black muck, roots, stumps, and trees than to leave them to vitiate the taste of the water afterwards, which they are almost sure to do. In constructing your dam, if of earth, carry your core wall down to an impervious strata in the bed of the valley. If this is not done, too much

water is wasted. Also under existing knowledge of the liabilities of all parts of the country to what are practically water spouts, more than ample provision should be made for storm waters by means of spill ways. The investigation of the disaster at Johnstown, Pa., showed the previously conceived idea of extreme flows must be considerably enlarged. In locating a pumping plant there are a number of points to be guarded against if the supply is from a river whose variation of height of flow is very large. If they are placed too high up no water can be carried through the suction pipe in extreme low stages, and if that point is taken care of there is a liability of the boiler and pumps being submerged in extreme high stages. Both of these contingencies have occurred.

In constructing a system for delivering by pumping, the question arises, how shall the water be handled, directly from the pumps through the mains on what is known as the Holly system, or by pumping into a reservoir or stand pipe? All engineers of good standing deprecate professionally, the Holly system, yet there are many cities supplied in this manner and with satisfaction. There is great economy in handling water by direct pressure as the consumers can only use what the pumps furnish. But the direct service is not so satisfactory, as time elapses between the notification of the fire to the pumping station the rise in the pressure necessary for the fire thus being done by putting on more power and this loss of time is often a potent cause of large conflagrations. On the other hand, pumping water into a standpipe or reservoir compels the pumping plant to deliver water at a constant head and the only extra demand is for more water in case of fire, which in a reservoir is generally at hand; and also there is no method of restricting the supply in cases of drought, as people living on the lower level will be wasting water, while those living on higher ground may be absolutely deprived of any supply.

With an ideal delivery of sufficient water at all times at a constant pressure ready for all emergencies, there can be no doubt that the reservoir or stand pipe practice is far preferable and I would so recommend it to you. Of course a stand pipe is only used when there is no available high ground for the construction of reservoir from which water may be delivered to all required points.

There is one advantage of a reservoir over a stand pipe and also over the Holly system, viz., that in many cases night pumping can be dispensed with and the saving of one engineer's pay is quite an item in small water works. With a stand pipe intermitting pumping is often resorted to, but in the Holly system the pumping must or should be

continuous to make the service effective. There is one other point I wish to allude to before closing my remarks on construction, and that is don't change the design and pattern of your supplies, having once adopted the kind of hydrant you think best or the pattern of gate; adhere strictly to that pattern all through your work as it simplifies materially the question of maintenance. The different firms furnishing water work supplies are composed of enterprising men, and one improvement adopted by one firm will soon be incorporated in them all, so that practically in any selection that is made very little disadvantage may accrue. I allude to it now as the liability of change comes during maintenance. The constructing engineer having finished his work leaves for other fields and the superintendent comes in. He may have his fads and preferences and prefers another hydrant or another kind of gate. He leaves and another change takes place till there is no uniformity in the plant and difficulties and troubles arise. To you who propose to be engineers and to you who may be superintendents I say don't.

Having cursorily glanced through the leading points of construction, we now face the question of maintenance, and while in all probability the engineering profession may not be so particularly interested in the maintenance of water works, yet probably in the future many may have such duties to perform.

The two salient points to remember in maintenance are, viz., a continuous supply of water to all requiring, and a remunerative return for the money invested whether by a municipal or a private corporation. In most cases the two are combined, but it is on record that the service has been sacrificed for monetary returns and that no profit is made through excess of consumption. Both conditions are erroneous and in the end have to be adjusted.

At first sight it would seem rather curious to suggest that the first care of maintaining water works is to give a continuous supply. Yet so many factors enter in to defeat such an end that it is much more trouble to accomplish such a result than may be generally supposed. There always is a liability for mains to break, for gate stems to twist off, for hydrant valves to become cut, for ice to form, and last but not least a shortness of water.

The contingency of these occurrences must always be guarded against and a water works superintendent must always be on the alert, both night and day. If a main breaks in the middle of the night with the thermometer 10 degrees below zero, all hands including the superintendent have to be on deck and tackle the break at once, and I have always found in

such cases that it is better to be on hand to encourage the men at work in such trying situations than to give your orders, and then go to bed.

When a break occurs in an ordinary lateral at night, it is probably better to shut the section off and leave the repairs till daytime, when work can be done so much the faster. Then comes the problem of a continuous drought, with a liability of a short supply and the heroic attempts of a superintendent to eke his water out to tide over the dry times and the continuous warfare between him and the consumers to attain such an end; the close study of the weather reports and the satisfactory relief when a storm does come. It may be said that prudence and foresight may guard against all such occurrences, but no prudence can foresee the breakage of a section of pipe that had been in use eight or ten years and then suddenly collapsing. No prudence can always pull the money out of stockholders pockets when they know that with a conservative use of water the plant and its appurtenances would give an ample supply. Then the pumps may suddenly give out and if there is not storage enough more trouble ensues, and the pipes may be frozen in places and then comes a wail of distress from the afflicted. So you can see that a water works superintendent has to be continually on the jump and never knows exactly what is going to happen.

Of course, in large cities with low pressure and ample supply of water, such liabilities are reduced to a minimum and I know of a small plant in a little village with a thousand inhabitants, where the supply is taken from a spring at the foot of a dam, and the power is furnished by a turbine pumping the spring water directly into the mains with a relief set at a certain pressure and absolutely the only care to that plant is to oil up the machinery once a day, taking a man's time for 15 or 20 minutes and the rest of the time it runs itself.

In a gravity supply without pumping, with plenty of water, the care is reduced to a minimum and running expenses correspondingly reduced. But when pumping has to be done, not only greater anxiety results, but the extra expense is also a matter of solicitude. In small plants it is better, if it is possible, to pump only in the day time; but in many cases this cannot be done and a day and night engineer have to be employed and then the problem of looking after the night man arises; he may go to sleep and he may feel angry if he is watched too close. So good judgment has to be followed in getting successful results. Then the problem of repairs to engines, the care of the boilers, the purchase of coal and oil, and all the other necessary requirements have to be considered and looked after.

Another thing faces the superintendent, and that is the service; one consumer complains that the pressure is too high and another will come along and say the pressure is too low. I had a case where a manufacturer had put in his factory what are known as automatic sprinklers to guard against fire. He came to me and said he was under contract to furnish 70 pounds pressure of water and I gave it to him and probably more. The next day another consumer who owned about 15 houses came rushing in and said that nine or ten of the boilers in the houses had burst, so I was between two horns of a dilemma and it took time and trouble to adjust the differences. Then there is the care of the fire service, to see that the hydrants work and that proper pressure is on all of them, and also to see that after a fire when the hydrants are closed they are not shut off with a bang, so creating a water hammer and thus breaking the main.

Then there is the supervision of the house service and the tapping of the mains to do. A system has to be devised if no meters are used, by which every house is carefully inspected, all connections, taps, and faucets recorded and rated, which also is part of the superintendent's work. And then there are the extensions to advise, recommend or reject, and put in if recommended. The effect of these extensions on the general system, as a pipe required to-day for a local sewer might in the future be part of a necessary main for some general service, and the question of interest here enters in. And last of all the care and watchfulness necessary to collect the rates. Water rates seem to be classed in the same category as pew rents. Everything else is paid but them. Then if the rents are too long delayed there is the alternative of shutting off the supply, and it has not been found neighborly to do this on washing day.

I want here to insert the statement that it is better to put in what is known as a curb box and gate to every consumer. This obviates the necessity, whenever a house has to be disconnected, of opening up the street and digging down to the main and shutting off the supply there.

Then there is the question of meters or no meters. Each plan has its advocates. Where no meters are used a house is rated according to the amount of accommodation it has for water service and the consumer can use as much as he pleases. These rates are based on the average experience in this country of water consumed. If the supply is by gravity and ample, this is probably the most remunerative; but when pumping has to be done and every gallon of water costs so much money the argument for the use of meters has great weight, and if there is a restricted supply they are absolutely essential.

Against the use of meters the argument is urged of the extra capital invested, the greater loss in collection from the fact that ordinary rates are payable and collected in advance; whereas, when meters are used the bill of necessity cannot be made out till the water is actually used and then also arises the friction between consumer and company of the correctness of the meter.

It may be, however, safely predicted that with the prospective desire for economy that will face the people of this country in the future, and the lack of desire by investors of putting any more money than can be helped in the extension of plants, that meters will come more and more into use. It is theoretically the true way, and with the advent of a meter in the market that shall be beyond cavil they will be generally used.

These remarks give you a rough outline of the general idea of water works. If they influence you to further investigation and study, they will not be made in vain, and in closing I wish to allude and warn you of one trouble that you will experience in handling work of this class.

The water work industry, if I may speak disrespectfully of a professional practice, is one in which is invested large capital and uses plenty of brains. Inventions are continually being made in this line both for the economy of construction and ease of maintenance. On the water work engineer and superintendent devolves the task of sifting, in all the appliances, the good from the bad, and on him will devolve the responsibility of such selection. Every known argument will be used to influence such selection and while of course error may be made, a conscientious judgment will at any rate be appreciated. And in conclusion let me remind you that Civil Engineering is a profession; that water works construction and maintenance is a branch of Civil Engineering; and that professional practices and standards are to be insisted upon in this work as in other branches of engineering. I allude to this because it has been whispered and suggested that the word commission has sometimes an influence in selections, and I wish to insist that no engineer should consider as a professional man anything except his fee or salary, and that the word commission be debarred from his catalogue of engineering terms.

Details of Construction of Engineering Structures.

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By engineering structures I have in mind those static structures, the design and construction of which come under the supervision of the civil engineer, such as buildings, bridges, viaducts and other similar structures. The materials principally used in the construction of the same consist of stone, wood or iron.

While the use of stone and wood for engineering structures dates back to antiquity, iron as a structural material is of quite modern origin and has almost superseded wood for permanent structures. Iron may therefore be considered the material of the future and must necessarily demand to a great extent the attention of the coming engineer. By iron I do not mean wrought iron, which is also a thing of the past, but ingot iron, commonly called steel. I will therefore confine my remarks to structures in iron.

It is not my intention to give you instructions in designing iron structures but only to offer a few practical hints on some important points which I think are oftentimes overlooked, but which have repeatedly come under my observation during my practice as an engineer, designing, inspecting and superintending the construction of iron structures.

It is a deplorable fact, that in many of the books on iron structures, more particularly in those that are written for the student and young engineer, some of the worst types of construction are illustrated ; types that would answer very well to show how structures should *not* be designed, and yet there is no intimation given but that they are examples of good practice. These bad examples, however, would be very instructive if they were properly criticised and attention called to their defects.

It is my hope that these brief, practical suggestions I have to offer may be useful to you in the future, and enable you the better to appreciate the good and the bad features of a design.

Those who do not possess in some degree that gift which is termed practical judgment or constructive talent, may by perseverance and study become fairly good designers; but their designs will be more or less imitations of the designs of others; they will be satisfied if their work is in conformity with the usual practice. The advantages of possessing this constructive talent are the same to the engineer, as the ear for music or the gift to distinguish the differences of sound are to the musician. Those of you who after some practical experience find that you do not possess the qualifications necessary to make a good designer, would do better to obtain employment in other branches of the profession to which you may be better adapted. However, I would advise everyone of you, if you have an opportunity to do so, to gain some experience in designing structures, as this experience will be of value to you some time or another, more particularly after you have advanced far enough to be called upon to pass judgment on the designs made by other engineers.

The selection or adoption of the right kind of structure, and the general outlines of the same for any particular case, to meet the existing conditions and best fulfill its purpose, requires experience and judgment and is generally left to the decision of older and experienced engineers and therefore does not come within the scope of this lecture.

In treating this subject of constructive details, we will consider:

1. The proportioning of the various members composing a structure and the designing of the details connecting the same; and

2. The preparation of the drawings for the instruction of those who are intrusted with the practical construction of the work.

PROPORTIONING OF VARIOUS MEMBERS AND DESIGNING OF DETAILS.

In order to proportion the members of a structure and their connections, we should keep in mind the following requirements, which should be fulfilled in a rationally designed structure:

1. The design should be complete in all its details; all the exterior forces which may possibly act upon the same should be considered, and provisions made to resist these forces.

2. The design should as far as practicable conform to the theoretical requirements.

3. The details of a design should be simple, effective and practical.

Sometimes we find structures which may be called incomplete structures, in the the design of which, some of the forces which may act upon the same were either neglected or no adequate provisions made to resist

or transmit the strains resulting therefrom. Occasionally we observe on structures which are exposed to wind and wherein the wind strains are important factors, such as sheds and buildings, that only the vertical forces have been considered, while the wind force in a horizontal direction is entirely or partly neglected. I know of some sheds of considerable size, in the designing of which, great attention was paid to the roof part of the structure; provisions being made for snow-load and the vertical component of the wind pressure, while the columns supporting the roof were calculated to resist vertical loading only, and the force of the wind horizontally against the sides and roof entirely neglected. Several disasters have been the result of these oversights. The same may be said of some high buildings constructed with an iron skeleton frame. If you see the iron framework of a building braced with temporary wooden braces during erection, you may be sure that there is something wrong; that something has been neglected. The iron framework of a building should at least be strong enough to resist the wind force on its own exposed surface, so as to be able to stand up during erection without apprehension. While it is true, that the addition of the stone, brick or terra-cotta work, which is carried by the iron frame, together with the partition walls, will act as a bracing in the completed building, they are uncertain quantites and should not be entirely depended upon.

In making provisions for the wind strains in an iron building, it is fair to assume that the walls will be of some assistance in bracing the structure, and, for this reason, it would be legitimate to allow a fibre-strain for the columns and braces of three-fourths of the elastic limit of the material, if the combined strains of vertical loading and wind pressure are considered.

Other illustrations of incomplete structures, or structures with organic defects, we often find in the lateral systems of bridges, where the bracing has been proportioned to resist the assumed wind pressure, but no provisions made to transmit the strains in the bracing to the chords of the bridge, which also form the chords of the horizontal wind truss, completing the lateral system. This remark applies to the lower lateral systems of most bridges which have the floor beams suspended from the pins and the laterals attached to the floor beams some distance below the pin; the floor beams acting as lateral struts. This construction is to be particularly condemned, if adjustable rods are used for lateral bracing. Similar defects have proved disastrous in many cases. If the floor beams are riveted to the posts above or below the pins and the laterals attached to one of the flanges of the floor beam, the lateral

strains are transmitted to the chords through the vertical posts, which will have to resist a bending strain. In such cases proper provision should therefore be made in the posts to resist the combined strains resulting from vertical loading and the bending strain from the lateral system.

Another case where one of the exterior forces is often neglected we find in plate girders which carry a railroad track on the upper flange,—where the number of rivets in the vertical legs of the flange-angles is just enough to transmit the horizontal shear between web and flange, the strains from the concentrated loads of the wheels of the locomotive having been entirely neglected.

In considering the exterior forces acting upon a structure, provision should also be made for such accidental forces as are likely to occur in the ordinary course of events and which might be foreseen or anticipated. It is for instance a common occurrence in ordinary railroad traffic for the truck of a freight car to derail, or for a broken axle or loose wheel to strike the web-members of a through bridge; hence, provisions should be made to meet such contingencies by firstly connecting the members so that they will not be knocked out of place by a blow from a passing object, and secondly by designing the sections of those members which are exposed to such accidental forces so as to offer some resistance against these forces. We often find that in small pin connected bridges the posts near the centre, (having barely section enough to carry the dead and live load strains), are so limber as to shake like aspen leaves when a train passes over the bridge, ready to yield at the slightest provocation. In a properly designed bridge, such small accidents may do some damage to the bridge, but should not cause a break down. I know of a bridge failure which was caused by a derrick-car. The derrick became unfastened and while swinging around struck some of the truss members thus causing the bridge to collapse. This accident could have been avoided if the design of the bridge had been carefully considered.

A roof over a machine, boiler, or blacksmith shop is likely to be put to some uses for which it may not have been intended originally, such as attaching blocks or pulleys to it for the purpose of lifting some heavy pieces.

A careful designer should forsee these things and make proper allowances for the same. While we should make provisions in our designs for ordinary contingencies, it is however, not possible to provide for all conditions, such as human forethought could not anticipate or those which are of such a nature that it would be impractical to provide for, such as tornadoes, unprecedented floods, earthquakes, etc.

We will now consider the theoretical requirements of structural details. It is not always the case that the most theoretical men make also the most theoretical designs. It is one thing to have the ability to analyze strains with great accuracy, and another thing to design the details in conformity therewith. The computation of the strains in the principal members of a structure is the least part of the work of designing; but the designing of proper details and connections is what requires the most skill, combined with good judgment and experience.

Let us now consider: what are the theoretical requirements of a design? Before the details are worked out, the design is drawn as a skeleton, the members being represented by lines; the strains in each member are calculated under certain assumptions, and the sections of the same proportioned for a permissible working strain to comply with the specifications. These permissible strains assumed in proportioning the members of a structure may be called the theoretical requirements. In order to meet these requirements, we should endeavor to design the details so that the exterior forces assumed in our calculations should produce strains in the members and their connections somewhat near the specified permissible working strains. Therefore, that structure in which the actual fibre strains in all its parts, come nearest to the permissible strains assumed in proportioning the same, is the structure which comes nearest to the theoretical requirements. In order to fulfill these requirements it is necessary to design the details so as to reduce the secondary strains to a minimum. Secondary strains in an irrationally designed structure sometimes increase the fibre strains in its members considerably over and above the permissible working strains prescribed in the specifications. In some cases as much as 25 per cent. or more is added by secondary strains.

I have examined existing bridge superstructures, which were designed for a supposed factor of safety of five, wherein the fibre strains in some places were so near the elastic limit of the material, that the factor of safety was reduced to a very small fraction, if the secondary strains were considered together with an unfavorable combination of circumstances such as is likely to occur within the limits of ordinary traffic.

In order to reduce the secondary strains to a minimum, our design should comply with the following conditions:

1. The neutral axis of all the members coming together at a connection should intersect in one point, as do the theoretical lines on the strain diagram.

2. All members should preferably have symmetrical sections.

The injurious effects of eccentric connections are too often ignored, as I will show you on several examples taken from the actual practice of the present day. Unsymmetrical sections in important members seem to be the rule rather than the exception. Compression-chords of pin connected structures often have unsymmetrical sections, the pin being placed in the middle of the section. The actual strains in a chord thus designed will be more or less at variance with the theoretical requirements, unless the strains produced by this eccentricity are considered in proportioning the sections. In some cases we find the pin placed in the line of the neutral axis of a compression-member of unsymmetrical section. Here the effect is similar to that of eccentric loading. The strains from the pin centres will have to travel unequal distances to reach the extreme fibres and will therefore be unequally distributed over the section, as the iron is an elastic material. Good practice requires that all members of a structure should be of equal strength and that the connections should develop the full strength of the body of the members connected. To make all members of a structure of equal strength would not be a very difficult problem to solve, at least approximately, if we had to provide for static loads only. However, as in most cases our structures have to sustain a live load besides a static load, this matter becomes more complicated, and it depends on the judgment of the designer, or on the one who prepares the specifications for the design, to accomplish approximately the desired result. Take for instance the floor of a ware-house, which is to be loaded with a certain class of goods; it is easy enough to determine the maximum load which may have to be carried on this floor; and as this maximum load is a static load, the strains resulting therefrom are well known. The case, however, is different when we consider the loading on the floor of a drill room, ball room or a highway bridge. All of these structures receive their maximum loading from the weight of a crowd of people. By ascertaining the number of people which could be crowded in the alloted space, we can calculate the static load resulting therefrom. But as these masses of people are moving, and more particularly if they keep time, with their steps, the effect of the static load of their weight will be increased by the vibrations resulting therefrom, which is generally called impact. In case of a floor of a building, the floor joists which receive the impact direct will be most affected; the girders which carry the joist will be less affected, and the columns which support the girders will receive a still smaller percentage of impact. As up to the present time we have no reliable data to determine this impact, it is left to the judgment of the engineer to in-

crease the live load by a certain percentage, for each case, to provide for the effect of this impact. The strains produced by impact in railroad bridges are even more uncertain and ambiguous. But until more reliable experiments have been made than we have up to the present day, to ascertain the effects of impact, we can only work with assumptions, making the effect of impact on the members of a railroad bridge dependent upon the length of loaded distance, which produces the maximum live load strain in a member, and add in accordance therewith a certain percentage to the live load strains to provide for this impact. In order to be on the safe side, we assume low working strains, generally less than one-half of the elastic limit. If we were able to determine the actual maximum fibre strains which could possibly in any case come upon a member of a structure, including the secondary strains produced by the deformation of the system, we could allow a working strain of three-fourths of the elastic limit of the material, which would yet leave ample margin for imperfections of material and workmanship. Some engineers vary the permissible working strains for different members of a structure in proportion to the minimum and maximum strains, still clinging to the superstitious belief in the fatigue of metal. All experiments of which I have any knowledge, and more particularly those made recently by Prof. Bauschinger, prove conclusively that the material does not get tired and that any repetition of strains below the elastic limit does no more injury to the material than if the strain is applied only once. Experiments on metal above the elastic limit have no practical value, as in practice we only work with strains below that limit.

If it is desirable to make all pieces in a structure of equal strength, each piece in itself should also be of equal strength throughout in all its details. As a simple example let us take a post or strut composed of two channels, connected by lattices or tie-plates. This strut is proportioned for a certain strain per square inch of section, depending upon the proportion of length to the least radius of gyration. Now, in order to keep the fibre strains within this limit throughout, it is necessary that in the sections between the connections of the lattice bars, the proportion of length divided by least radius of gyration of one channel section, should not be greater than that of the entire strut. The same applies also to the ends of posts, which are sometimes made weak by cutting away a portion of the flanges. This is often done by unskilled designers on vertical posts of pin connected bridge trusses, thus producing a weak point, where good practice requires additional strength, on account of the strains from the lateral system, which

generally have to pass through the ends of the posts, (which have already been weakened by the pin hole), to reach the chord system. If you will examine some of the existing structures designed in that way, you will find that the ends of the posts in most cases are bent or distorted, which has been done either during transportation, by handling or by driving the pin during erection. You will, therefore, see that a neglect in this respect may have serious consequences.

In order that a connection may develop the full strength of the members connected, it should be so designed that the strains therein can be readily analyzed. Complicated contrivances will generally not admit of being satisfactorily analyzed and should, therefore, be avoided. Among the grim-crack connections which should be avoided, may be mentioned—wing plates screwed on the ends of pins, U nuts, rods with bent eyes, loops, stirrups, etc. Other weak details which do not develop the strength of the members which they connect, are frequently encountered in connections between riveted girders, such as the connections between longitudinal and transverse floor girders of a bridge. The ends of the longitudinal girders generally have connection angles riveted on, reaching over the vertical legs of the flange angles with a filler under them between the flange angles. In many cases this filler is not wider than the leg of the connection angle, which makes the rivets fastening this angle to the web plate of very doubtful value, as the rivets will have to resist a bending strain. This bending strain increases with the thickness of the filler. In order to give the rivets their full value, the filler should be wider than the angle and have enough rivets outside of the angle to make the fillers a part of the web plate. The same criticism applies to many similar connections of which this one is only an isolated example.

Good practice also requires that the details should be simple, effective and practical. To design the most simple details may appear to be an easy task for those who are inexperienced in designing, but as a matter of fact, they require the most skill and study. It took many years to develop some of the simplest and most effective details which are now used in good practice. In order to make a connection simple and effective, the strains should be transmitted from one member to another in the shortest and most direct way. Simplicity of details and connections should be studied not only for the purpose of obtaining the best and most effective construction, but also with regard to economy of shop work and to facilitate erection.

In order to make a design practical, we should in the first place consider the work in the shop; make designs which will simplify the shop

work, and reduce the cost and select such details which will produce correct work. That is, the shop should have no difficulty in producing the finished work precisely as shown in the drawings, so that the pieces will properly fit together and the whole structure may be erected without trouble. If we expect the shop to do precise work, all pieces should, as far as the nature of the structure will admit, be straight, without bends, twists or crooks. Avoid blacksmith-work should be our rule. If, however, the nature of the work is such that blacksmith work cannot be avoided, consistent with good construction, and it should become necessary to heat and bend some pieces, it should be so arranged that a piece has a bend in one place only. This is easily done by pressing the same in a former of the proper shape. If there are a number of bends in one piece, which cannot be made in a single operation, the work becomes not only more expensive, but also inaccurate, as we have to depend upon the skill of the workman to obtain the desired shape.

For the rationally designed structures, the operations of the workshop are reduced to shearing, punching, riveting, planing, turning pins and rollers, boring pin-holes, and forging eye-bars. All contrivances which require skillful forging by hand are impractical. Among the impractical contrivances may be classed all adjustable members, such as rods with screw ends, loops, turnbuckles and clevises, as are frequently used for laterals and sway braces in bridges, viaducts and buildings. Adjustable members are impractical, not only because they are expensive to make, but being composed of screw ends, loops, clevises or turnbuckles, are less reliable than plain substantial work. In time they become loose and out of adjustment and have to be readjusted frequently, which is very undesirable in a permanent structure. They are flimsy and unsubstantial, and on account of their rattling, when used in a bridge, during the passage of a train, may be termed rattle trap construction. Wherever you find a rattle-trap structure you will generally find gim-crack details and connections accompanying the same. The ideal structure has no adjustable members, and when once erected needs little or no attention, excepting a coat of paint once in a while.

When structures are proportioned,—that is, the sizes and sections of the members determined upon to conform to the strains,—the arrangement of the details and connections should be considered at the same time. That this is not always done, I am reminded by a paper on bridge details which was published in one of our engineering journals a short time ago. The author of the paper was an engineer of one of our prominent bridge companies. In this paper among other things our engineer complains that the ingenuity of the detailing engineer was

sometimes greatly taxed in trying to work out the proper details and connections of a structure and at the same time conform to the general proportions of the members as given him by the strain-sheet maker. This appears to be the way designs are manufactured in some of our engineering shops. However, this is not the way the work should be done in a *well regulated* engineer's office. The work of proportioning the members of a structure previous to making the detail drawings of the same should be done by men with good judgment who are experienced in making detail as well as shop drawings and who are acquainted with the shop practice. If, during the designing of the details, it should be discovered that, with the given proportions, good details cannot be designed, the general proportions or sections of some members should be changed so as to admit of good and effective details and connections. It is to be regretted, that as a general rule so little attention is paid to the structural details of designs for engineering structures.

Railroad companies are the largest purchasers of iron structures. It is their usual practice to let the contractors for the iron work make the plans of their structures to conform to their engineer's specifications. Before the contract is awarded to the successful bidder, the strain-sheet submitted with the bid is generally referred by the Chief Engineer of the railroad company to one of his younger assistants for the purpose of verifying the strains and sections of the proposed structure, to see if the same conform to the specifications. This strain-sheet fiend generally complies with the request of his chief with a vengeance, confining his investigations to an elementary consideration of the strains; these he figures to an ounce, and calculates the sections of the members to an infinitesimal fraction of a square inch, but pays little or no attention to the real design. The more complex questions of details and connections are generally left to the contractor who sometimes keeps a stock of stereotyped, antiquated details on hand, ready to be furnished to his customers on short notice. As our calculations of strains are at the best only rough approximations, as far as the actual strains of the live loads are concerned, you will see the absurdity of too great refinement in one respect, and, on the other hand, the criminal carelessness with which the details and connections of a structure are often neglected. Similar absurdities of too great refinement in the wrong place, we often notice in specifications for structural work. Some specifications make fine distinctions between compression members with pin ends and with flat ends, allowing a greater working strain for flat ended struts. If any distinctions are made, they should be made in favor of the strut

with pin ends; giving it a larger working strain as the pin ended strut receives all of its strain concentric, while the flat ended strut is generally subjected to eccentric strains because of inaccurate connections, or distortion of the structure.

Let me admonish you to consider designs of structural work in the light of reason and common sense, not only elementary, but also scientifically as well as practically.

We will now consider:

THE PREPARATION OF DRAWINGS FOR THE INSTRUCTION OF THOSE THAT ARE INTRUSTED WITH THE PRACTICAL CONSTRUCTION OF THE WORK.

If all engineers understood how to make drawings properly and more particularly the young men who have just graduated from an engineering school, it would be useless to say anything on this subject. But as experience has taught me that the young engineer requires generally a good deal of instruction and admonition before he is able to make a drawing which is up to the standard of good practice, I am impressed with the necessity that this subject should be given more attention than it usually receives.

In making drawings we should generally follow the rules of descriptive geometry, with the exception of some conventional signs, which have been universally adopted and are therefore sanctioned by usage. The drawings should be clear, the lines uniform, neither too heavy nor too fine, but prominent enough to be plain and distinct. It is also advantageous to have the sheets of uniform size and the drawings made to a uniform scale, as much as practicable. The lines of the drawing proper should be black; full lines show that they are visible, while dotted or broken lines represent those which are invisible. Centre lines, dimension lines and construction lines should be in red. Cross-sections are usually indicated by diagonal parallel lines (hatching); the lines to be drawn in different direction for adjacent pieces, in order that the different pieces may be more readily distinguished. If the sections are very small, they may be filled up solid with a small space between the adjacent pieces, so that each piece may appear distinct. If a cross-section cuts through a pin, bolt, rivet or similar piece of cylindrical shape, those pieces should not be shown in section, but in elevation; as it will add a great deal to the clearness of the drawing by showing these pieces more distinctly, besides saving the labor of cross-hatching. All unnecessary work which does not add to the information given by the

drawing should be avoided; for this reason nuts and bolt-heads should not be shown in plan, but only the bolt or pin holes should be shown. As these have regular standard sizes, it would be superfluous to show the same on the drawing, excepting in some cases where they may be shown in elevation. The dimensions should be written in plain figures, the dimension-lines having an arrow point at each end to indicate the extent of the dimension. All writings should be clear and plain; no ornamental or fancy lettering should be allowed on any drawing; but always bear in mind that the writing primarily is there for information and not as an ornament. The title, scale and date should be in plain letters in the lower right hand corner of the sheet, in order to facilitate the finding of any drawing when filed in a drawer or piled up on a table.

The above remarks apply to all drawings for structural work. I will now take up shop drawings particularly.

Shop drawings are made for the instruction of workmen and should therefore be made with this end in view. The drawing of each piece should be made so plain and complete, that the shop men may easily understand it without asking for explanations, and to enable them to find all dimensions and obtain *all* information necessary to do their work. A judicious arrangement of shop drawings will greatly facilitate the work in the shop, reduce the cost of the shop work, and prevent mistakes which would occur if the drawings were not properly made. This subject is therefore of such importance as to demand our attention. In order to make a shop drawing which will fulfill its purpose, it is necessary to be acquainted with shop practice, which may vary slightly in different shops. However, the general principles which should govern the making of the drawings will apply to all shops. The lines of a shop drawing should be of uniform thickness; shade lines denoting projections do not seem to add anything to the clearness of the drawing. My experience has taught me that the workmen are better able to read a drawing without shade lines, as they do not understand the meaning of the same. They only make additional work without improving the drawing as far as its usefulness is concerned.

End views, or cross-sections, should be placed at the ends which they represent looking toward the piece. The top view should be placed above, and the horizontal section showing the bottom, below the side elevation of the piece. The bottom of a piece should always be shown as seen from above and not as seen from below, so as not to be confusing to the workmen. As many views and sections should be made as is necessary to show any part of it clearly and plainly, and no

more. If two views of a piece show every part of it, it is not necessary to have any more. For example—if a drawing of a riveted piece, such as a top chord of a bridge, or a plate-girder, shows all the rivets in the upper flange on the top view of the piece, it is not necessary to show the same rivets again in the side view or elevation; nor is it necessary to show the latticing of the bottom flange of that chord piece in elevation, when it is shown on the longitudinal section of the piece. Repetitions of this kind fill up the drawing unnecessarily and, instead of adding information, produce confusion, while at the same time a good deal of time has been wasted to no purpose. Long members may be shortened by omitting intermediate pieces, if the same contain only a repetition of rivet spaces or a series of lattices of equal spacing. In accordance with the nature of the piece, one or more breaks may be made.

The different pieces composing a structure should preferably be drawn in their relative positions which they will occupy in the structure. Horizontal members, such as chord pieces of a bridge, should be drawn horizontally; vertical posts, vertically, or if more convenient to draw them in a horizontal position, the lower end should be on the left side. Diagonal members may, for the sake of convenience, be drawn horizontally, the lower end always on the left side.

All shop drawings should be made to a scale, as this is sometimes useful to check the dimensions, and it also shows the correct proportions of the various pieces and gives the workmen a good idea how the finished piece will look. Every piece should be designated by a mark corresponding to the mark on the general drawing and erection diagram. The drawing should give the number of pieces required for each member. If a number of pieces are similar, but reversed, belonging to opposite sides of a symmetric structure, they will all be made from the same drawing; a note saying so many pieces right and so many pieces left, or in pairs when right and left are of equal number, is sufficient and will be understood by the workmen. The sizes of all the pieces composing the member should be put on the drawing in such a way that they can easily be found. The drawing of each member should have a bill of material alongside of it. This bill of material is a tabulated list of the different pieces of material shown on the drawing as composing the member, giving their sizes, shapes and finished lengths. The bill of material is put on the drawing in order that the different pieces forming one member can be picked out from the material received at the shop from the rolling mill without much trouble. The pieces are marked with the sizes given in the bill and the mark designating the member to which they belong.

All *dimensions* should be written on lines which are projected off from the centers to one side, or better outside the piece altogether, so as to leave the drawing as clear as possible, and the figures written between the projection lines. Arrow points should be placed at the points between which the distances are given. No dimensions should be written on a center line of a piece, nor the rivet spacing on the center line of a row of rivets. Only necessary and exact dimensions should be given; dimensions which are of no particular use or those which are not exact should be left out. The filling up of a drawing with a lot of superflous dimensions is confusing to those who have to use the same, and the time spent in writing these dimensions is wasted. In order to know which are the dimensions which are needed, you must think while you are making a drawing and follow in your mind the different manipulations a piece of work has to undergo during the process of construction and anticipate the needs of the workmen. We will now take the example of a girder consisting of a web plate and four angles, being the most simple case which could be selected for an illustration to more fully explain what I have said before. The drawing should, of course, contain the size of the plate and the sizes of the flange angles, rivet spacing, size of rivets, etc. In order to make this girder you must bear in mind that the holes in each piece composing the same have to be punched or drilled separately; then the pieces are assembled by bolting them together temporarily by a few bolts, and then the rivets are driven. The sizes of the web plate and the angles being fixed as well as the depth of the girder,—what other dimensions should the drawing contain in order to be useful for the shop? The holes in an angle are always measured from the back of the angle and not from the end or inside of the leg; because the angles as well as other shapes are not rolled with mathematical accuracy; and as we want to maintain the depth of the girder, no matter how much variation there may be in the sizes of the angles, you will see the necessity of giving the distance of the guage line for the rivets from that point. The only other dimension necessary is the distance between the two rows of rivet holes for the web plate. In this case, therefore, the only dimensions necessary for the guage lines of the rivet holes are distance back of angles to center of rivet and distance between upper and lower row of rivet holes. All other dimensions would be more than useless, because they would be inaccurate.

The principal dimensions of a piece should be put on the drawing first, and given separately and distinctly from those minor dimensions, such as rivet spacing, or spacing of lattice bars, tie plates, etc. Let us

take for an example a top chord piece of a pin connected bridge. The distance from end to centre of pin hole, and distance between centres of pin holes, should be given separately, and should not be written on the same dimension line on which the rivet spacing is given for the rivets in the pin plates; but these rivet spaces should always start from the center of the pin. The same refers to the spacing of the lattices and and tie plates on the bottom flange of this top chord section; they should not be given as a continuous series of figures from end to end, but should always be referred to the pin centres, these being the important dimensions in this case. This should be done for three reasons: firstly, that the dimensions may be easily checked; secondly, to enable the men in the shop to lay out their work intelligently; and, thirdly, that the important and exact dimensions may be obtained from the drawing directly, without adding up a number of minor dimensions. All the figuring and calculating should be done in the office, and not in the workshop. In making a drawing of a riveted joint, where several members connect around one point, the centre lines of rows of rivets in the connecting pieces should intersect in a fixed point, if possible, on the centre line of the principal member, or on the center line of a row of rivets in the same; and this point of intersection should be fixed by a rivet, in order to give the templet-maker a precise point to work from. The angles of the connecting pieces with the principal member should be given as the sides of a right-angled triangle; but not in degrees, minutes and seconds, as is sometimes done, as the workmen are not supposed to be able to solve problems of trigonometry. However, in the facing of pieces at an angle, which is done on a rotatry planer, the angle should be given in degrees and minutes. The drawing of any piece which is connected with other members in the structure should have the ends of the connecting pieces drawn in with red lines. This is done, firstly, for the convenience of the draughtsman, so that he can make his designs of that piece to suit these connections, to show him where rivet heads which would interfere may have to be countersunk, flanges to be cut away in order to clear one of the connecting members, or to show him the positions of tie-plates, etc., and, secondly, that the drawing may be checked more readily and also enable the men in the shop to do their work more intelligently.

Duplications:—In order to still more reduce and simplify the work of making drawings, as well as the work in the shops, we should aim to make as many pieces alike as possible, or use one drawing for a number of similar pieces. You will find in most structures many pieces which are, in their general features, alike, excepting a few minor details.

For all these pieces the same drawing should be used, by drawing only such portions which are different, so that the same templets with slight changes may be used over again for pieces which are almost similar.

Conventional Signs:—In order to give proper directions for riveting, it is necessary to have some conventional signs to denote the different kinds of rivets; indicating which rivets have to be driven in the shop, and where rivet holes should be left open, and the rivets driven "in the field;" whether the rivets have full heads or countersunk heads, also calling the workmen's attention to those rivets whose heads may have to be flattened. There are now several codes of rivet signs in use, but some of them are either too arbitrary, or not distinct enough to be satisfactory. These conventional rivet signs should be such that they can be easily remembered and easily made, and should be distinct. I have adopted a code of rivet signs which have proved very satisfactory in practice, which I will now describe:

Rivets are usually indicated by a circle of the diameter of the rivet-head, and open rivet-holes by a circle of the diameter of the rivet hole blackened. If you want to show a countersunk rivet-hole, it would be natural and rational to show the same as it appears to the eye, viz: with two circles, one of the diameter of the rivet blackened, and the other one of the diameter of the outer edge of the countersunk hole. If the countersink is on the near or visible side, this outer circle will be in full line, and, if on the far side, it should be drawn in dotted lines. Applying this same principle to shop rivets, we simply leave out the blackening of the inner circle which denotes the open hole, and we have the sign for countersunk shop rivets; if the countersink is on the far side, we use a dotted line for the inner circle. We will now need another sign denoting that the rivets are countersunk on both sides, for which we will use a diagonal cross, it being easily made and easily distinguished. Sometimes rivet-heads may have to be flattened, generally in cases where one member fits into another, and the available space will not permit a full-sized rivet-head. In order to draw the shape of the flattened rivet-head in plan, we would have to make a double circle, the same as for the countersunk rivet. We will therefore have to use a different sign to denote flattened rivet-heads. As they are very seldom used in good practice, they are not of as great importance as the signs for countersunk rivets. As a flat is sometimes represented by a short heavy line, it would make a good sign to show flattening by drawing a heavy black line diagonally across the rivet-head; this line being dotted will denote that the flattened head is on the other or far side; and two heavy lines crossing each other at right angles mean that heads are flattened on both sides.

I should also mention in connection with this subject of conventional signs, that it is customary in some offices to shade the surfaces of cuts, such as cuts of flanges of angles and channels, which is very useful, and good practice, for calling special attention to these points.

Standards :—In order to have some uniformity in the shop work, it is necessary to have a system of standards, which are used by every engineer in the office to assist him in making detail and shop drawings. This system of standards, however, should not be carried so far as to include details of construction, but should only cover such general points, as the rivet spacing for different shapes and sizes of rivets, the sizes of eye-bars, pins, nuts, lattice bars, clevises, turnbuckles, upset ends of rods, beam connections, etc. Standards have the advantage besides that of making the work more uniform, of relieving the draughtsman of considering these little points everv time he makes a drawing, which would take a good deal of his time. These standards have generally been worked out by experienced men for the benefit of those employed in the office, and will therefore be productive of better results than if every one, and more particularly those of little experience, would decide upon the dimensions, etc., which are given by the standards. This system of standards has been in some cases abused by including details of construction. This I consider bad practice, as it prevents the designer from exercising his own judgment and ability and from making improvements. The designer simply becomes a machine which manufactures designs. I know of cases where complete standards for constructive details were marked out for all the details of railroad bridges of different spans, under the supervision of the engineer in charge of the office, thus making machines of his assistants, who were simply manufacturing designs by joining together a lot of ill advised details, the results of which consisted in a lot of structural monstrosities, samples of which are yet on exhibition on some of our railroads.

In concluding my remarks on drawings for structural work, I would take the liberty to make a suggestion, referring to the instruction of students in the way of making drawings somewhat similar to the way they are made in actual practice. Instead of stretching a piece of paper on a drawing-board, let it be simply tacked down with thumb-tacks, make the drawing in pencil, and then make a tracing of the same. This will give the student an opportunity to get some practice in making tracings, which would be of some advantage to him when he enters practical life in the profession.

Notes on the Design and Construction of Masonry Dams.

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I shall not occupy your time on this occasion with an extended discussion of the theory of masonry dams. This part of the subject is well worked out in the various text-books, and inasmuch as it has attained so far as our present knowledge of the laws of mechanics goes, apparently the fixed state, nothing further can be added thereto, and our time may be more profitably used in jotting down such information as the ordinary text-books have thus far mostly failed to furnish.

Undoubtedly, the theoretical character of the discussion of masonry dams, as found in nearly all the text-books, has been a real difficulty in the way of the beginner. There has been a disposition frequently, to consider the design of dams as complete when the profile was determined. So marked has been this tendency that the title of a recent paper before the Institution of Civil Engineers was "The Design of Masonry Dams," although, as pointed out in the discussion, the paper was really confined to suggestions for facilitating the calculation of the profile of a dam on the ordinary hypotheses set forth by the French engineers, and by Rankine. The important questions of foundations, regulating works, waste-weirs, and the cognate questions derived from a study of the regimen of streams, are left entirely untouched. Obviously the title should preferably have been, "The Computation of the Profile of Masonry Dams."

Mr. Wegmann's book on "The Design and Construction of Masonry Dams," while probably the best text-book of the subject thus far published in any language is still open to the criticism that it deals largely with the profiles—one only gathers from it incidentally that regulating works and over-flow weirs are indispensable adjuncts of such a construction. It follows then if one desires information as to the design of the necessary adjuncts, it must be sought widely in official reports

and the proceedings of technical societies. As assisting such research I have given at the end of this paper a list of the various publications I have had occasion to consult, together with a brief statement of the information to be obtained from each.

One may properly remark in passing, however, that the comparatively recent working out of the rational theory of the profile of masonry dams is, to a great extent, a sufficient reason for this remissness of the text-books. Still in view of the enormous saving of material effected by the use of the so-called economical profiles it is not strange that the methods of deducting them should have been accorded a prominent place.

Again, the failure of a large storage dam is of such serious import, that we find an additional reason why discussion of the profile has thus far been justly deemed of first importance. Still, mere mass in a reservoir wall cannot be taken as a certain index of security. The Puentes Dam, in Spain, failed eleven years after its completion, from a defect in the foundation, which, instead of being carried to the bed rock, had been erroneously made on a platform of timber work supported on piles. This dam had a profile section far in excess of any possible requirement and its failure may serve to teach that a high masonry dam is only safe when founded entirely on rock.

And right here, following the thought of Sir Benjamin Baker, in the discussion before the Institution of Civil Engineers just referred to, it may be pointed out that however valuable the purely theoretically discussions of the proper form of profile may be, there is still an element of uncertainty in the results attained, due to the nature of the assumptions necessarily made, and although mathematicians may properly suggest formulæ, it is still in the end for practical men to decide whether or not the formulæ may be safely accepted.

The construction of high masonry dams for the purpose of impounding large bodies of water is not a modern idea. In Spain there are several high dams, among the chief of which we may mention the Almanza dam, with an extreme height of 68 feet, and the Alicante dam, with an extreme hight of 135 feet, both of which have been in use since the latter part of the sixteenth century.

There are several other Spanish dams of considerable age, but none of them are in any degree models to follow at the present time. They all show a profile far beyond any necessary requirements. We know nothing of the methods used by the Spanish engineers, who designed these early dams, to determine their proportions. Probably no special method of analysis was used, but each engineer constructed what

seemed to him to be a perfectly safe structure. A few Spanish dams constructed in the last twenty years are, however, designed on the lines of the modern economic profiles.

But it is to the French engineers that we are indebted for a rational solution of the problem of high masonry dams.

The first writer who investigated the theory of such dams in a satisfactory manner, was Mr. De Sazily, who published a memoir on the design of reservoir walls in the Annales des Ponts et Chaussees for 1853. He formulated the two following conditions for safety of a masonry dam:

(1) The pressure sustained by the masonry on the foundation must never exceed a certain safe limit.

(2) There must be no possibility of any portion of the masonry sliding on that below, or of the wall moving on the foundation.

Mr. De Sazily also pointed out that in determining the maximum pressure two extreme cases must be considered:

(1) When the reservoir is full.

(2) When the reservoir is empty.

The next discussions on the line of the modern economic profile were those of Mr. Graeff and Mr. Delocre, which appeared in the Annales des Ponts et Chaussees for 1866. These engineers designed and constructed the Furens dam in accordance with the views announced by Mr. De Sazily, and improved upon by them in the design of this dam.

In 1870 Professor Rankine was called upon to report on the construction of a proposed dam for the water supply of Bombay, India. In his report he suggests a type profile, and calls attention to a number of points in the design of such profiles, not noted by previous writers.

Mr. Bouvier also discussed the design of such profiles in an article on the Ternay dam which appeared in Annales des Ponts et Chaussees for 1875.

There are a number of other papers dealing with the design of the profile, of some interest, which are not referred to here for lack of space, and we may complete this part of the subject with a reference to a paper by Professor Frantz Kreuter, of the Royal Technical Academy of Munich, on The Design of Masonry Dams, to be found in Vol. CXV of the Proceedings of the Institution of Civil Engineers. This is the paper referred to in our opening paragraphs.

According to Professor Kreuter, the basis for any sound theory of masonry structures may now be stated as follows:

(1) That, at any horizontal section, the intensity of pressure at the faces of the wall shall never exceed a certain value fixed upon as the safe crushing load of the material of the dam.

(2) That at no horizontal layer of the masonry shall there be any danger of sliding.

(3) That at those parts of the profile where the wall has a batter, the intensity of pressure at the faces shall be diminished below the limits answering to vertical faces.

(4) That there ought to be no practically appreciable tension at any point of the masonry, whether at the outer face, when the reservoir is empty, or at the inner face, when it is filled. The lines of resistance therefore should not deviate from the middle of the thickness of the wall to an extent exceeding one-sixth of the thickness.

Professor Kreuter makes the following assumptions:

(1) The water level is supposed to reach the top of the wall.

(2) The vertical component of the water pressure upon the battered part of the inner face of the wall is provisionally neglected.

(3) The shearing-stresses acting parallel to the layers of the wall are not allowed for.

The first and second of the above assumptions are made to simplify the calculation, although they at the same time favor safety. The third is of no importance when the wall is properly executed, according to the rules established by the earlier writers.

Professor Kreuter's paper is so lengthy that, all that can be done is to refer to it here. In the discussion, Mr. Wegmann, whose work on "The Design and Construction of Masonry Dams," has been already referred to, criticised some of the methods of the author as leading to an unnecessary mathematical refinement which could not be realized in practice. The whole discussion may be read with profit by any person interested in the design of such dams.

The profile having been determined, the next question for the engineer to decide will be as to the plan of the dam, whether it be straight or curved.

Dams of both classes have been successfully built, nevertheless we find a nearly unanimous consensus of opinion, that if a curved plan be adopted, it is still advisable to so proportion the profile as to enable it to stand, by its own gravity, the pressure of the water.

If, however, the dam is proportioned to withstand the water pressure by gravity, it is difficult to see how it can act in any degree as an arch. On the other hand, if the section is not sufficient to withstand the water pressure, then the dam must yield under the strain and some pressure will be transmitted to the abutments at the sides of the valley. It appears, then, that the two conditions of withstanding pressure by gravity, and at the same time acting as an arch are incompatible. The two cannot act together at the same time.

The following are some of the more interesting dams which have been made curved in plan: The Furens dam at St. Etienne, France, built to furnish a water supply for the city of St. Etienne, and power for manufacturing purposes in the river valley, is curved in plan with a radius of curve equal to 827 feet. This dam is also so designed as to resist the water pressure by gravity purely. The Zola dam, near Aix, France, built for water supply, water power and irrigation purposes, is curved in plan, with a radius of 158 feet. The Bear Valley dam in California, built for irrigation purposes; is curved in plan with a radius of 300 feet. The Sweet Water dam also in California, and built for irrigation; is curved in plan with a radius of 213 feet. There are a number of other curved dams in different parts of the world, but the foregoing are enough for present illustrative purposes. The three last mentioned, Zola, Bear Valley and Sweet Water all acting as arches, are insufficient to resist water pressure by gravity alone. Of these the Original Bear Valley dam is by far the boldest structure of the kind yet erected anywhere. It relies so entirely upon the arch action that one cannot help thinking, in view of the little we know about arches, that it is a dangerously unsafe structure, although one must not overlook that it successfully withstood the maximum pressure possible to be brought to bear upon it, without showing so far as known any signs of weakness. Possibly it is a case where success is a sufficient answer to any and all cavils. On the other hand, the fact that a new dam of the usual economic gravity section has been recently erected at a location, 150 feet below the original one, may be taken to indicate that the designers had some doubt as to the safety of the first dam.

There are, however, certain reasons why the curved form is desirable: namely, it will adapt itself more readily to changes of volume, due to changes of temperature. The various questions raised under this head were gone into extensively by Messrs. Joseph P. Davis, James R. Croes and William F. Shunk, who, as a board of experts examined the plans of Quaker Bridge Dam, and reported thereon to the New York city Aqueduct Commissioners, under date of Oct. 1, 1888. These gentlemen reached the following conclusions:

(1). That in designing a dam to close a deep, narrow gorge, it is safe to give a curved form in plan, and to rely upon arch action for its stability; if the radius is short, the cross-section of the dam may be reduced below what is termed the gravity section, meaning thereby a cross-section or profile of such proportions, that it is able by the force of gravity alone to resist the forces tending to overturn it or to slide it on its base at any point.

(2). "That a gravity dam built in plan on a curve of long radius, derives no appreciable aid from arch action, so long as the masonry remains intact; but that in case of a yielding of the masonry the curved form might prove of advantage.

"The division between what may be called a long radius and what may be called a short radius, is, of course, indefinite, and depends somewhat upon the height of the dam. In a general way we would speak of a radius under three hundred (300) feet as a short one, and one over six hundred (600) feet as a long one, for a dam of the height herein contemplated.

(3). "That in a structure of the magnitude and importance of the Quaker Bridge Dam, the question of producing a pleasing architectural effect is second only to that of structural stability, and that such an effect can be better obtained by a plan curved regularly on a long radius than by a plan composed of straight lines with sharp angular deflections.

(4). "That the curved form better accommodates itself to changes of volume due to changes of temperature."

Messrs. Davis, Croes and Shunk, also included in their report discussion of other interesting points, as, for instance, in regard to the influence of the thrust of ice upon the design of structures of the class now under consideration. Without giving their reasons therefor, they express the opinion that the Quaker Bridge Dam, should be proportioned to resist an ice thrust at the highest water line of about 43,000 pounds per lineal foot.

Undoubtedly the most important point to be attended to in the construction of a masonry dam is as to the foundation, since without a perfectly safe foundation all other precautions may be futile, and the dam liable to fail. We may lay it down then as a fundamental proposition, that a high masonry dam should always be founded on solid rock and every precaution taken to remove all danger of leakage under the dam, as otherwise there is a consequent liability of its destruction. Moreover, we must assume that a masonry dam is an absolutely rigid structure, and the least unequal settlement will tend to produce cracks. The examinations of the foundation must, therefore, be of the most searching character, and if anything other than perfectly solid rock is found to exist, the whole site subjected to the closest scrutiny before beginning to build. This proposition is especially enforced when we remember that the loads on the lower courses of a high dam may amount to from 10 to 15 tons per square foot, and it is unsafe to superimpose such a load upon anything but the most substantial rock.

The design of high masonry dams exhibits in its various aspects

many interesting problems in physics, some of which were presented so thoroughly in the discussion of Professor Kreuter's paper before the Institution of Civil Engineers, to which we have already referred, that we can hardly do better than refer to them a little at length here. For instance, Professor Forchheimer called attention to the fact that varying temperature and the penetration of water into the interstices of the masonry naturally exerted a tendency to disruption, which thus far had not been taken account of in any computations. As regards the penetration of water, it might be assumed to flow in approximately horizontal lines through the masonry, and as its velocity was constant it would suffer a uniform loss of pressure, and this pressure multiplied by the volume of the interstices per unit volume of the mortar would give the up-lift. Still the safer plan is to make the inner face of the dam as water-tight as possible. In a dam lately built in Remscheid, Westphalia, the inner face was plastered, and then rendered over with two coats of asphalt. He also suggests that greater security could be gained by draining the wall so constructed close to the inner face.

As regards the movement due to temperature, Professor Forchheimer considered this of even greater importance, especially in countries subject to climatic extremes. It had been observed by Professor Intze that at the middle of the crest of the Remscheid dam, which was 82 feet in height, there was a backward and forward movement amounting to 1 1-16 inch during filling and emptying of the reservoir, and that the movement due to temperature was almost as great as this. The crest of this dam is 460 feet long and arched in plan, with a radius of 420 feet, one side of it is exposed to the sun longer than the other, and the more exposed part has been observed to move to and fro 7/8 of an inch in the course of a year, while the other part moves only 1/8 of an inch, the crest expanding 5/8 of an inch. Professor Forchheimer points out that the great advantage of arched dams is that they permit of these changes taking place without harm, while in straight dams the same phenomena become very objectionable, and that there might be considerable danger of fracture of a straight dam, as shown by practical experience. The dams of Habra, Grands-Cheurfas and Sig, in Algiers, had broken, and in that of Hamiz a tear had occurred during the first filling. The Habra dam broke in December, and the Grands-Cheurfas and Sig dams gave way in February. The Beetaloo dam, in Australia, had also developed a crack 1/8 of an inch wide, in the middle of the winter, and without any apparent cause. In the winter of 1890-91, in a period of extreme cold weather, seven vertical cracks appeared in the Mouche dam, a structure 1,346 feet long, and about 100 feet wide.

These cracks were widest at the top, and situated at nearly uniform distances apart of about 160 feet; they died out about 37 feet below the normal water level. Their aggregate breadth was 2⅞ inches. As the temperature rose, they gradually decreased and in a short time four of them had completely vanished, whilst the other three had preceptibly contracted. The opening and closing of cracks at different seasons has also been noticed in other masonry structures, and a long quay wall at Bremen is cited which developed cracks during the periods of low temperature, of 1-6 and ⅓ inch, and which closed to fine hair cracks in summer.

On the other hand, it seems doubtful to the present writer whether if properly constructed, any such extreme changes as are here mentioned would have happened, and the statements of Mr. Deacon, the engineer of the Vyrnwy Dam, of the Liverpool Water Works, in the same discussion may be cited as showing that at any rate, the question is open to discussion. Mr. Deacon began by saying that there is a small longitudinal tunnel passing through the mid-section of the Vyrnwy Dam at a distance of rather more than 80 feet below the sill. When the dam was in process of construction certain stones were bored vertically as they were laid, one above the other, so that a bore hole about 4½ inches in diameter passed from the top down 80 feet to this tunnel. From the top of this bore-hole, a steel piano-forte wire was suspended, carrying at its lower end in the tunnel a large weight immersed in water. Near the bottom was an apparatus of the seismograph kind by which any movement was multiplied four times. A fine needle moving without friction recorded the magnified movement. Unfortunately no satisfactory records were obtained until the water had reached within 13 feet of the sill, but the movement during the rise from the 13 foot level to the top water level was such that the sill moved horizontally with respect to a point in the dam 80 feet below, to the extent of 0.868 of a milliimetre, or about one-thirtieth of an inch. Changes of temperature caused the dam to move from night to day, and from day to night. The dam faces nearly southeast, so that it does not receive full heat of the sun, but when the reservoir is full the maximum horizontal movement of the sill with reference to a point in the dam 80 feet below, is as between a hot summer day and a summer night 0.360 of a millimetre, or about one-seventieth of an inch. These movements are not only much less than those recorded by Professor Forchheimer, but they have not been attended by any visible disruption. Taking everything into account the Vyrnwy Dam is perhaps the best constructed work of that character thus far carried out anywhere, a fact which very

likely has been of material influence in preventing any harmful result from these inevitable movements, due to changes of temperature. The small movement due to rise of water may be considered as resulting from excessive strength of this dam, and it accordingly becomes an interesting question, whether after all, in localities subject to extreme climatic changes it is on the whole desirable to rely too much upon the strict theoretical economic profiles, but rather to give dams some excess of section as a pure matter of insurance.

The use of curved dams by increasing the length of the dam adds somewhat to the expense, but the advocates of such dams claim that we have in this form a practical solution of the problem which we are now discussing. At any rate, whatever conclusion you may arrive at when you are called upon to design high masonry dam, you may take this part of the subject as illustrating the truth of the proposition that in this, as in all other classes of hydraulic works, it is not best for the engineer to struggle too much for economy.

The quality of the material of which high masonry dams may be constructed has been a subject of considerable discussion among engineers. Generally speaking, the French dams have been constructed of small rubble stones, such as two or three men can easily handle, and the French engineers have made elaborate arguments to show the snperiority of this class of stone for such work. In the Furens dams, the stones varied in volumes from about 2 to 7 cubic feet, but in the Vyrnwy Dam the opposite extreme was gone to, and stones were used varying in volume from 1 to 4 cubic yards.

Masonry dams may also be built of cut stone masonry and of concrete, although the great cost of cut stone would ordinarily prohibit its use. Again, inasmuch as the form of the upper part of a dam depends upon the positions of the lines of pressure rather than on the strains in the masonry, the excessive strength of cut stone masonry would only become available in the lower courses of a dam. Its use would therefore lead to an unnecessary excess of strength in the upper portion of such a structure.

A further disadvantage of the use of cut stone masonry throughout the whole extent of a high dam, would be the tendency which it would have to slide if laid in horizontal courses, and great care would accordingly become necessary to insure a proper degree of irregularity in the courses if it were used. In order to prevent the passage of water directly through the courses, it is necessary that neither the vertical nor the horizontal joints be continuous. In any case the joints should be carefully broken.

Cut stone has sometimes been used as a facing for dams constructed with the interior portion either of rubble or concrete, the intention being to give the outside portions where the greatest strains are encountered an excess of strength. In some cases cracks and seams have been formed, because of the difference of settling of the two kinds of masonry. If such settling should proceed to the extent of detaching the facing from the balance of the wall, the strength of the structure would be reduced in a corresponding degree. The following are a few examples of the use of cut stone facing with rubble or concrete backing.

(1) The Betwa Dam, in India, has its faces constructed of coursed dimension stone surmounted by an ashler coping. The backing is uncoursed rubble throughout.

(2) The Boyd's Corner Dam, on the West Branch of the Croton river, in this state, was built with cut stone facings with the interior of concrete, the lower portion having large stones bedded in the same.

(3) The new Croton dam now constructing, for the water supply of the city of New York. This dam is of composit construction, partly earth and partly masonry. The overfall-weir is of a heavy cross-section and 1,020 feet in length on the crest. There is also a masonry section of a length of 630 feet, designed on a theoretic profile. These two portions are to be constructed with coursed faces, with backing of uncoursed masonry.

(4) The Tytam Dam, in China, is perhaps as interesting an example of this form of construction as any. The total height of this dam is 120 feet, the top width 23½ feet and the width at the base 62½ feet; an extension of 10 feet in the height at some future time is provided for. The foundation block is of rubble concrete, with cement rendered face. The interior of the dam, or the backing, is of rubble concrete with fine and extra-fine concrete skins; the upstream face is of ashler, and the downstream of rubble masonry.

The ashler of the upstream face was laid in one-foot courses, alternately headers and stretchers, the latter being about four feet long, and the former one-foot square, and one-foot nine inches in length; the tails of the headers and backs of the stretchers were left rough. All the joints were tuck-pointed with a flat joint three-fourths of an inch wide, as the work progressed. The down-stream face is of rough quarried rubble stones, laid one stone thick, the concrete backing holding the stones in place. The joints were pointed with neat cement, though numerous holes were left in the pointing for drainage.

The mass of the dam is composed of granite and rubble stones imbedded in concrete.

A number of other dams have been constructed with the facings of somewhat different material from that of the interior, and although such form of construction is not approved of by many engineers, there seems to be no doubt but that it can be made satisfactory, provided sufficient care is taken in all the details.

Concrete has also been used in the construction of a number of the largest dams yet constructed, as, for instance, the San Mateo Dam, in California, 170 feet in height; the Periar Dam, in India, 155 feet in height; and the Beetaloo Dam, in Australia, 110 feet in height.

Concrete may be considered as rubble masonry reduced to its lowest terms, and one great advantage derived from its employment, is that, ordinarily the expense may be lessened by reason of the saving of labor due to mixing and handling by machinery. When well laid, concrete may be considered as quite as impervious and nearly as capable of resistance to crushing as ordinary uncoursed rubble masonry, and there is no reason for supposing that any greater care is required in order to secure good work by the use of the one rather than the other.

When large streams are to be dammed, the design of the spill way or overflow-weir becomes a problem of far reaching import. For its solution we must know the maximum flood-flow of the stream, and usually in the absence of direct gauging giving this fact at once, we need to consider the probable maximum run-off as derived from a study of rainfall records, and the results obtained from lysimeters, or drain-gages. It is accordingly seen then, that the proper design of the spill way will include a wide range of investigation.

In a recent discussion before the American Society of Civil Engineers, Mr. E. Sherman Gould, gave a formula for length of spill way in terms of the drainage area expressed in square miles. But, as I have endeavored to keep formulæ out of this paper, I will merely cite you to it as being an attempt to apply computation to the solution of this problem. My own view is that the length of spillways is not susceptible of numerical computation from the data which Mr. Gould uses, and indeed in justice to that gentleman it must be stated that the formulæ is not put forth as anything more than a very rough guide.

As illustrating the importance of a thorough understanding of this part of the design, it will suffice to mention that the new Croton Dam, with a drainage area of 340 square miles back of it, is to have a spill way 1,020 feet in length, but it should be understood that this spill way has been increased in length considerably by reason of discharging over the face of the dam.

In those cases where it is necessary that flood-waters be discharged

over the front of the dam, it frequently becomes desirable to build some form of protection against the erosive action of the falling water at the toe of the dam. For this purpose water-cushions have been extensively used in India, with good success. They consist of a subsidiary weir across the channel at some distance below the main dam, and of such a heigth as to make a pond below the main dam deep enough to effectually counteract the scouring tendency of the downfalling water. Mr. Wilson, in his report on irrigation in India, which appears in Part Second of the Twelfth Annual Report of the United States Geological Survey, has given a formula for depth of water-cushions. Like all such formulæ, it is very general in its application, and judgment will be a much safer guide in the construction of such works, than adherence to the formula. In case there are any natural falls upon the stream to be dammed, or in the vicinity, and which are also of the same geological formation, intelligent observation of the depth of the pool formed naturally at the foot of such falls, will be a much safer guide than the application of any formula whatever.

As an interesting example of a dam in this country with an efficient subsidiary water-cushion weir, we may refer to the Turlock dam, in California. This dam is straight in plan, 310 feet in length on top, 96 feet wide at the base, 20 feet wide on top and 130 feet in height. It is estimated that the flood-waters of the Tolume river will pass over the crest at a possible depth of 16 feet. About 200 feet below the main dam is built a subsidiary weir 20 feet high and with a top width of 12 feet. The rock foundation on which the dam stands is said to be hard and firm, and a water-cushion of this height successfully resists erosion.

The foregoing gives but the barest skeleton of what appears to the present speaker so be the more interesting debatable questions relating to masonry dams, a subject by far too extensive to be treated satisfactorily within the limits of a single paper.

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Wilson, Herbert M.—(1) * Irrigation in India, Part II of Twelfth An. Rept. U. S. Geolog. Survey, (1890–91). Gives an account of several of the high dams of India, with statements of cost, quality of material, etc. The most of what little we know of water cushions in this country is derived from Mr. Wilson's report. (2) Irrigation in India. Trans. Am. Soc. C. E., Vol. XXIII, (1890). A monographic condensation of the elaborate report to the Geological Survey. (3) * American Irrigation Engineering, Part II of the Thirteenth An. Rept. U. S. Geolog. Survey, (1891–92). Contains description of the principal dams for irrigation purpose in the West. (4) American Irrigation Engineering, Trans. Am. Soc. C. E, Vol. XXV, (1891). Monographic condensation of the proceedings. (5) * Manual of Irrigation Engineering. 8vo. New York, 1893.

In addition to the foregoing a large amount of valuable information about high dams may be found in *The Engineer*, *Engineering*, *Engineering News*, and *The Building Record* for the last ten years.

Restriction of Immigration.

By Gen. FRANCIS A. WALKER,

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It should be said at the outset that what we are to talk about, is not keeping out a certain number of the insane, criminals and paupers, such as, from lack of proper legislative measures, have found their way in great numbers to our ports in the past and have mingled with the body of our citizenship, much to our injury. No one questions the propriety of excluding persons of unsound mind or vicious habits, or persons incapable of caring for themselves or supporting themselves. That, therefore, is not under discussion. Our government has, as I have said, been very lax in regard to this matter. Thousands and tens of thousands of the classes referred to, have come here, perhaps sent purposely by their governments or municipalities in order to get rid of them. A great deal of that sort of material has in the past been dumped upon our shores; but now that public attention is fairly aroused upon the question, there can be no two minds on the subject of the exclusion of such people. What we are speaking of is not keeping out some thousands who cannot take care of themselves; it is the exclusion of hundreds of thousands, all of whom, we may assume for the purposes of this discussion, are capable of supporting themselves. They are not criminals nor paupers, or vicious persons especially; they are, we may say, able-bodied and capable of self-support, at least according to the standard of their native lands.

It must seem strange to any person brought up in this land of ours, to hear such a question discussed at all. All through our history, from the earliest time, there has been much felicitation over the extent to which immigration has risen, and the height at which it has maintained itself from year to year and generation to generation. Immigration has been regarded as one of the sources of our national strength. I can remember how, a score of times, the newspapers have rolled the

last figures of an unprecedented immigration, like a sweet morsel under their tongues; and congratulated their readers and the nation upon the extent to which the access of persons from other shores and climes has been carried. So that, if it is a question which it is worth while for intelligent people to consider, whether we shall now put up the bars and keep out the tens of thousands, the hundreds of thousands of persons who are yearly coming to us, or at least restrict very greatly their number, it certainly exhibits a great change in public sentiment—a change so great as perhaps to excite incredulity. If we were right then, must we not be wrong now? If we are right now, must not our fathers have been very weak, short-sighted and foolish to felicitate themselves on the access of foreigners?

I can only say that we have abundant instances of changes of opinion and changes of attitude on the part of our people and of other peoples, changes quite as decided as in this case, and yet fully justified by changes of condition which had taken place. I might mention many instances, I will only take one. I select that one because it seems to me to bear a close analogy to the case we are now considering. Take the matter of the cutting down of our forests. From the first settlement of this country, until a very recent period, the Pioneer's Axe has been, more than any other material thing, the emblem best fitted to be the symbol of American civilization. The axe of the pioneer has been celebrated in song and verse, and sober prose. We have all listened with admiration and with delight to the story of the march of that great army of axe-men westward, levelling the forests as they went, opening up the soil to cultivation, clearing the ground to found school houses, churches and colleges. Yet to-day we hear men say that the denudation of the soil has gone beyond what is safe and desirable; that the cutting down of our forests should be restricted. The government of the United States even offers premiums, large premiums too, for the planting of trees upon our western lands. Here is a change which is quite as marked as that which is immediately involved in our present discussion, yet a change the justice of which is fully recognized. The time was, and that for a long period, when the cutting down of our forests were really and truly a subject for felicitation. The time has, however, come when the further destruction of our forests is threatening the greatest dangers to American agriculture, to American meteorology and to all the material interests of this country. So it may be that our fathers were fully right in taking the position they did regarding the arrival of foreigners upon our shores, and in felicitating themselves upon the wonderful development of

immigration through many years; and yet it may now be reasonable and right that we should look upon the continuance of immigration with distrust, apprehension and alarm.

Before proceeding to the positive part of my subject the direct argument as to the restriction of immigration, it seems to me desirable to look at two opinions respecting the immigration of the past which have been held almost without dissent by the American people, yet opinions which, in my humble judgment, are wholly erroneous. I can hardly speak of any other political or economic opinions which I believe to have been so entirely unfounded as those to which I now allude.

One of these opinions was that immigration constituted a net reinforcement of our population; that, the more foreigners came, the more we were in numbers, population being increased by something near, if not by the whole sum of those who entered the country. This opinion was universally held by our fathers, and I think is very greatly held to-day. As increase of numbers was then a relatively important matter, in the condition in which our fathers found themselves—with the country partially settled, with enormous resources undeveloped, with possibilities of unbounded wealth before us—immigration was welcomed, and it was mainly welcomed because it was believed to constitute a net reinforcement to our population.

I believe no opinion ever held widely by an intelligent body of people was so entirely unfounded. Look at the facts of the case. From 1790 to 1830, forty years, the American people increased at the rate of from thirty-three to thirty-eight per cent every ten years, a rate of increase never, in the history of human population, maintained, over any considerable area, for any considerable time, except here in the United States. That tremendous increase was entirely out of the loins of our own people. Immigration at that time scarcely existed at all. All the foreigners arriving between 1790 and 1830 constituted but an inconsiderable addition to our population. The American people, mainly a native, and wholly an acclimated population, increased at a ratio which has never been approached, so far as I know, on any other soil and under any other climate, in human history. Had the rate of increase in the population from 1790 to 1830 been continued to the present time, we should be to-day 92, or 94 or 95 millions of people, instead of perhaps 67 or 68. In other words, we have fallen short of making good the promise of those first forty years, by twenty-five millions.

What was the cause of this falling off? Why was it that, so long as we had a purely native population, we maintained this astounding rate of increase of population? Why was it that a decline in the birth-rate

coincided exactly with the beginning of the large immigration? For that coincidence existed. Year by year as foreign immigration increased, the rate of the native increase declined. It fell off in the very years in which the foreigners began to arrive in greatest number; it fell off in the very regions and sections of the country into which the foreigners came most freely. A more exact coincidence statistically is hardly to be found in human history; and, for one, as a life-long student of population, I have no doubt that it was the very incoming of the foreigners which checked the rate of the native increase. Instead of foreign immigration being a net reinforcement of our population, it was merely a replacement of native by foreign stock, a replacement which, after a while, ceased even to make good the traditional rate of increase of the native population.

The whole history of human population shows how intensely sensitive to social and economic conditions the principle of population is. Let those conditions remain unchanged, and population will go on increasing, generation after generation, like gas expanding in a vacuum, at a rate which you can predict with almost absolute precision. The incoming of the foreigners, out of the peasantry of Europe, largely degraded through no fault of their own, and accustomed to the most squalid and miserable conditions of life, constituted a great shock to population in a country where social ambition had always been active and where the standard of living was higher than in any other country of the world. The new-comers naturally fell into a social and industrial class by themselves. It was the first time since the settlement of the colonies that distinct social classes were known, at least here at the North. Such a fact was of itself a sufficient reason, in the view of any vital statistician, of any student of human population, for the shrinking-back of the natives from such a contact, such a competition, such an association. Then it came about that no American father would feel that his life had been anything but a failure if he had to bring up his sons to labor on equal terms and an equal footing with those who had entered the country in this way. I do not desire to speak unkindly. It was no fault of these people that they came out of such homes as they did; and that they came with vastly lower ideals of social decency and with habits of living repulsive to the native American. It was not at all their fault; yet none the less did it have this effect upon our population. That effect was a perfectly normal one. The cause mentioned was entirely competent to produce such an effect, in kind and in degree. I have no doubt that, instead of the great immigration of that time being a reinforcement of the population

it was, first, a replacement of the native population by foreign stock, while, after that, it even failed to make good the loss of native population.

It was certainly no decline of the vitality of the American people after 1830, in their reproductive power and general vigor and length of life, which caused this falling off in the native increase. After 1830 the old manslaughtering medicine, from which the American people had suffered more than any other people on earth, went out and the new medicine came in. The people had better food, better clothing, and better housing. The change in the American physical type since 1830 has been most favorable, even remarkably so. We have here a soil and climate which are highly favorable to length of life and to physical vigor. We have taken the European Short-Horn and improved it so that a cow has been sold from Western New York, to be re-exported to England, for $45,000. We have taken the English race-horse and developed it until to-day no horses that run on English turf are the equals of our Kentucky horses. We have taken the English man and made him the best sprinter, the best shot with either pistol or rifle, the most effective football rusher, the most dangerous prize fighter, the best all-round athlete of the world. The statistics of height, chest measurement and weight of our American army during our late war, when hundreds of thousands were enlisted and registered, in comparison with tens of thousands of Englishmen, Irishmen, Germans and Frenchmen, show that, in all the elements of physical force, the American is to-day the best human animal in the world. It was no decline in the vitality of the people which caused the effect noted. On the contrary, the typical Yankee, long, lank, cadaverous and dyspeptic, has almost entirely disappeared from America and is only seen upon the stage. To-day our people are better fed, better nourished, have more life, more "go" in them, and more vital force and power, than in 1830. And yet this decline in the native population has taken place. I believe it to have been wholly or mainly in consequence of the incoming of the foreigner.

There is a second opinion regarding foreign immigration which prevents our taking a fair view of the present situation. It is an opinion almost universally held, yet one which I believe to be entirely mistaken. It is to the effect, that, whether the immigration in the past was in fact a reinforcement of our population or not, and whether the social and political effects of immigration were good or bad, it was necessary that the foreigner should come to us in these great numbers, in order to do a certain kind of work which the native population would not do.

That argument has been used a thousand times in the past thirty years. We did not greatly relish having our population made up in this way; but we submitted to it because the native American would not work in the sewers, in the canals, on the railroads; in a word, would not do the lowest kind of manual labor, and so the foreigner came to take his place.

Here is another opinion which illustrates admirably the mistaken policy of putting the cart before the horse. When did the native American first refuse to do any kind of manual labor? In the last century? No. In the early part of this century? No. Lawyers, doctors, professors, judges, governors, then did all about the house, or on the farm, which it was necessary for anybody to do. There was no such thing known as indisposition towards manual labor. You can find many old enough to remember when no man was esteemed too good or fine to do any kind of work which his hands found to do. When did the Americans first refuse to do mere ditching and trenching? I answer, when the foreigners came. As long as each man in his place did whatever was required of all men, there was no obliquy, no discredit, no shame about it; every man was willing to do that part which fell to him. But when we had a class in this country which could do that kind of work and nothing else, then the American ceased to do that kind of work; then he set himself apart; then he refused to join the gangs which were working on the canals and railroads. When manual labor became a sign of a want of education, want of training, and want of breeding, then every man who had any choice whatever, any control of his fortunes and destiny, drew out of it.

We have had a very curious caricature of this argument during the last few years. It used to be said that the Irishman came because the native American would not dig in the trenches. Within the last two or three years, *Harper's Weekly* had a very curious article about the incoming of the Italians. The Editor admitted that these did not constitute a very desirable addition to our population. As an element they had, perhaps, an injurious social and political influence. Still it was necessary to have them; the work had got to be done, canals must be dug, and railroads built; and, consequently, since the Irish were now generally refusing to do the very lowest kind of manual labor, we must have the Italians; and thank God for them.

You have only to see the argument thus in its second generation, so to speak, to detect the fallacy of all this sort of talk about the necessity of having uneducated and unskilled foreigners, with lower tastes and aspirations and meaner habits of life, come to America, to do the poorest kind of manual labor. Your grandfather and my grandfather, your

grandmother and my grandmother did a great deal of what would be called to-day, manual work. And they were not ashamed to do anything and everything which would minister to the welfare of their families in sickness and in health. They called nothing common or unclean. But when a class of people came in who could do manual labor and could do nothing else, then the native American drew back and left it to these people to do all of this sort of work, alone. And now it seems, after the lapse of thirty years, the Irishmen, in turn, think themselves too good. They are turning over the lowest kinds of manual labor to the Italians; and the Editor of *Harper's Weekly* cannot sufficiently admire the wisdom and benevolence of Providence in sending along the Italians to do the work which was thus in danger of being left undone. But did the Italians come because the Irishmen refused to dig in the trenches? No, the Irishman refused to dig in the trenches because the Italian came. Here is simply another instance of putting the cart before the horse. The in-coming of a class of people lower in respect to strength, energy and intelligence, has put the Irishman upon his mettle, and he, too, draws aside. In five years more, if Baron Hirsch should send us over two or three million of Russian Jews, you will find the Italians beginning to put on airs, drawing themselves to one side, and declaring that they will not do any more of mere manual labor. But what about our citizenship all this time? What of the Republic?

Ladies and gentlemen, I have dwelt so long upon these familiar opinions regarding foreign immigration in the past, because, until we get these out of our minds, we cannot fairly look at the question as it presents itself to us now. Yet, the question to-day is a very different one from that which our fathers contemplated. Great social, industrial and political changes have taken place in the last forty years. It is in considering these that I come to the positive argument of the case.

In the first place, the arable public-lands of the United States are entirely exhausted. Fifty years ago, any one who desired to cast in his lot with us, had no difficulty in finding public lands of excellent quality, which he could make his own by paying $1.25 an acre, under the Preemption Act, or even obtain free under the Homestead Law, paying merely the small fee for registration. We had then an enormous body, not merely millions of acres, but hundreds of thousands of square miles, of the best arable lands in the world, open to intending citizens. Our laws invited foreigners to come upon these lands, to enter themselves as citizens upon the registers of the nearest land offices, and to build homes for themselves and their children. During all that time, while there was an indefinite, a practically unlimited, supply of free public

lands of high quality, almost any amount of foreign immigration could be safely endured, because it was so easy for the foreigner to get upon the land. Once upon the land, the problem was solved. With a favorable soil and climate, it was his own fault if he failed to do well. It was in this way that a dozen great states were built up. So long as that condition lasted, the problem of immigration was very simple.

To-day there is not a single acre of land that any body knows about, fit to be taken up, which is not taken up under the Homestead and Preemption Acts. Of course, the United States Government still holds hundreds of thousands of acres of land, but they are in the Great American Desert, or form the rocky ridges of the great Appalachian chain. They are not worth taking up at all ; or, at any rate, they can only be taken up in large bodies by great capitalists who have the means of constructing comprehensive irrigation works. Perhaps I cannot call your attention to this feature of the situation more strikingly, than by reference to two recent experiences. You remember how, a few years ago, Oklahoma was opened to settlement and its lands taken up. There is no reason to believe that the lands were any better than the lands of Wisconsin, Iowa, or Minnesota, which had been taken up under the old system. But the peculiarity of the situation was that there was no other public land in the United States, fit to be taken. You remember, perhaps, how twenty or thirty thousand squatters lined-up on the very border of Oklahoma ; how cavalry had to be deployed along the frontier, with their carbines cocked, to prevent intruders from rushing over the line and making their way to the lands about to be opened. You remember the race which followed the firing of the signal gun, and the many amusing and terrible experiences of the next few days, while this army of "boomers" were filling up Oklahoma. You know that we have since had the so-called Indian strip opened up to settlement, when the same phenomena were repeated.

There is a second important condition in respect to which there has been change ; and that is in regard to the prices of agricultural products. You know that, for the past twenty years, there has been a steady decline in the prices of all agricultural products. Wheat and every other of the great crops has gone down, down, down, with no reaction, and apparently no promise of reaction. Instead of wheat selling now at \$1.12 in Chicago, or \$.90, it has gone down to \$.60, and below. The prices of agricultural produce have fallen lower than ever before and they persistently keep low, with no sign whatever of reaction, except possibly as the result of a "corner" gotten up by a conspiracy of speculators. Now, part of this, of course, has resulted

from a diminution in the cost of production ; but it is not wholly that ; it is mainly due to other elements. I need not go into the discussion of these. Among them is the competition of new countries, like India and the Argentine Republic ; but, to whatever cause this effect is due, it is a tremendous fact, because it diminishes the ability of the farmer and the planter to employ large numbers of partially skilled laborers upon the field, at high wages. Not only so, but it reacts upon manufacturers. It has been from the first, with us here in the United States, the competition of the farm with the shop which has kept up the price of mechanical labor. It was impossible for wages in our shops, factories and mills to be crowded down, by any cause, below a certain point, since there was this free access and these high prices for agricultural products, which justified a large employment of labor and high wages in agriculture. Thus we have had a strong support for the wages of manufacturing labor of the east in the high prices for agricultural labor at the west. Especially was this a means, and a most effective means, of taking care of the vast body of laborers arriving year by year in our country. But in this a change has taken place, and taken place, as we must believe, once for all.

There is a third change, also, a very serious change in our national conditions ; and that is, we have now a labor problem. The people of the United States, and especially our newspapers and representatives in Congress, have been very much given to taunting other nations with certain great advantages enjoyed here ; with our social peace and good order, with the fact that it was not necessary for us to maintain a large force of soldiers and policemen, because our people were so quiet and law abiding. All this was trne, but it was no great credit to us. We had not, then, a labor problem. The country had not been sufficiently exploited to induce that state of things when a painful congestion of labor, here or there, over wide districts, would be likely to occur at any time. That condition is permanent and chronic in Europe. There is hardly a country in Europe which has not long had a labor problem ; in which the limit of the demand for labor was not so nearly reached that, at any time, an unfavorable cause would produce this state of the market, which induces congestion, with resulting social disturbances, disorder, and danger. Bnt a labor problem has at last come to us, also. We shall do well not to boast too much about how easily our people govern themselves. We have now a market for employment which is so nearly glutted that we have become subject to these great disturbances ; and no country in the world is less prepared to deal with social disorder than the United States of America. We have not the machinery,

we have not the inherited ideas, we have not the instincts and traditions, for dealing with that kind of thing. And please to observe that, when your market is not filled, when you are rapidly spreading over a new country, a very large access of labor, even if not called for, might not cause any great difficulty. But when the market is well supplied, then the access of a certain amount of labor, even if it be not a very large amount, if unskilled, if there be not intelligence, enterprise, mobility among the new comers, may produce disorder and have a most serious effect upon employment and upon wages. It will be seen that this is a very important consideration and it puts the problem of immigration in a very different light, because, as you know, nearly nine-tenths of the laborers coming to us from foreign countries are unskilled. They are coming to a market already well supplied, which cannot take on new elements with anything like the freedom of the olden time.

The changes in condition, of which I have thus far spoken, are all subjective; that is, they relate to our capability of taking care of a large immigration, as compared with the capability of the country forty or fifty years ago. I have mentioned three points in which we are less prepared to deal with foreign elements than before. There is another change, and that is one which concerns the immigration itself; it is objective; it has to do with a change in the condition and character of the immigrant class. Of this I shall have to speak with perfect frankness, if I am to tell the truth and do justice with my subject. In the old days there was a pretty strong presumption in favor of the man who came to this country across the Atlantic, that, at least, he was one of the more energetic, ambitious, courageous, prudent and forethoughtful of the population from which he came. This might not mean a very great deal; but the immigrant had to have more than the average prudence of his village; more than the ordinary forethought, courage and intelligence of his own people, to find out that he was not well off where he was. There are some populations so downtrodden that they do not even find out that. Then, he had to have the mental inquisitiveness and intelligence to find out where he would be better off. He had to have the moral courage to cut himself off from everything dear to him, all the associations of his childhood and of his family. He had to seek his way across half a continent, perhaps, on the other side, and then across a wide ocean, to find a home here. The men who came in the olden time were, therefore, presumably among the more thoughtful, intelligent, enterprising, courageous, and aggressive of their stock.

How is it to-day? The change in conditions has been so great that the presumption is absolutely reversed. So widely has the net work of

European railroads been spread, so great has been the reduction in the cost of transportation by land and sea, so completely is the "drumming" service organized all over Europe for promoting emigration, that to-day it is among the most thriftless and shiftless the emigration agent finds his best recruiting field. It is among those who have not been getting on very well, who have not much courage or force of character; whose habits, perhaps, are bad, or who for some other reason are less fortunate than their fellows:—these are the people who are put on the cars and locked in by the agents of the Red Star or Cunard lines, upon the plains of Hungary or Russia, or France or Germany, or wherever it may be—consigned to the proper agent at Antwerp or Liverpool. At the ports, as I myself have seen, the doors are unlocked; these people are taken out and led in droves to the warehouses where they can lay themselves, their effects, and their babies down upon the floor, to await the sailing of the steamer. It is Pipe Line Immigration. It flows by gravity. And there is no reason why this should not go on from bad to worse, until every foul standing-pool of population in Europe shall be drained off into the United States.

So much is true of the old countries, whence immigration has for a long time come, countries that were familiar to the passenger-lists of the steamships of fifty or thirty years ago. But there is a worse feature, still, in the immigration of to-day. That is found in the presence among our immigrants of almost a predominant number of persons who come from nations and races that were not known to the immigration of fifty years ago, the nations of Southern and Eastern Europe. I am not speaking, I hope, in unkindness. We have got to face the facts. We are now receiving hundreds of thousands, fast becoming millions—who represent the worst failures of civilization; who have none of the political instincts, none of the ideas and habits of self-government which characterized the original settlers of this country or the early immigrants into this country from Western Europe. They are beaten men from beaten races. They are coming here with no political knowledge, no industrial education; with nothing like an industrial history behind them. They are untaught peasantry, who hardly know the use of the simplest tools, who have lived in filth and under the most unfortunate conditions possible on their native plains and who are transported hither in the way I have described.

There may be those who take such a thing lightly, who can say, "Oh, no matter, in a short time we shall make them just as good citizens as if they came of Teutonic stock; representatives of the men who met under the oak trees in the forests of old Germany to make their laws and choose their chiefs two thousand years ago." To this

I answer, ah, well, perhaps you can breed the instincts of self-government, self-respect, self-support, in a few years; but if you can, what is your civilization good for? If the instincts of self-support, self-respect, self-government, can be developed so quickly, they may disappear just as quickly. The belief we hold to is that self-government, self-respect, self-support represent the breeding of ages; that there are centuries and centuries on our side, as there are centuries and centuries against the men who are coming to us at this time.

Now, ladies and gentlemen, I have spoken of changes in our immigration within the last thirty or forty years—changes which affect us and our capability of receiving large numbers of immigrants, and also of a change among the foreigners themselves, as regards their degree of preparation to become good citizens and useful workers here in the United States. I have spoken on this subject too often not to feel that there is all the time a force working against me in the mind of every native American who hears me,—and I thank God for it,—which renders it very hard to make any progress in the argument of this question. I have spoken of practical considerations; but there is no people on earth so sentimental as the American people, no people where considerations of that nature have such great power. We have shown many times and in many ways the force of sentiment over our nation and our people; and, I am glad it is so. Now, all the sentimental considerations on this point oppose themselves to the restriction of immigration. We have been proud, as a nation, that we could hold the ports open, keep the flag flying, and make America the refuge and asylum for the oppressed and down-troden of every land and clime. We have boasted of it, perhaps in something like vain glory; but we have also been glad of it unselfishly, because it has warmed our hearts to think how much good we were doing to these hundreds of thousands and millions of people coming hither to make a home among us. I would not have Americans entertain such feelings less warmly or less strongly. I have no sneer for sentimental considerations like these.

Yet, if there is indeed danger to the republic from this access of foreigners—for remember that during the ten years between 1880 and 1890, five and one quarter millions arrived upon our shores—if, I say, in this enormous, overwhelming access of foreigners—now temporarily, during our own hard times, checked, but certain to spring up anew with increasing power—if there is, in this, danger to the nation, we have no right to allow ourselves to be governed by the feelings of which I have spoken, no right to make sentimental considerations in dealing with this question, predominant. The man who cares not for his own household is a heathen man and an infidel. Self-defense is the

first law of nature and of nations. If our country and its institutions are really imperilled, as many believe they are; most gravely imperilled, as we think we have seen them within the last few years; brought by this very cause, into dire danger and drawn almost to the brink of a precipice—then we have no right to refuse to consider the question in its whole length and breadth. We are bound to take it up seriously. If the interests of the country require it, we have no right to refuse to put up the bars, and to impose at least a temporary check upon this access of the most degraded elements of Europe to our shores.

But, after all, it is not you or I, or all the men who are fortunate in their own lives, who will decide this question. It is not what economists may think, or statisticians may think, or politicians may think; it is what the working people think, that is going to decide the question. I said that the American people are wonderfully sentimental; and it is also true that the most sentimental part of the population is found among our working people. Consider how ready the ordinary working man is to submit to privation; to go out on a strike; to see his family almost starve under his eyes, if he believes that the interests of his class require it. How few men among the more fortunate classes would you find making equal sacrifices. The working people of this country are going to settle this question of immigration. Just so long as they are willing and prepared to say: "We see the danger, we already appreciate the loss in the breaking down of the rate of wages, as in the clothing trade, in the tobacco manufacture, in the mining industry; we know there is danger to the American rate of wages; we see that the American standard of living is threatened by this in-coming of the foreigners; we would that it were otherwise; but we will not, being ourselves less fortunate than we could desire, knowing what the hardships of life are, we will not close the door upon any man who comes here in good faith to make a home for himself and his children after him. We will face all the dangers; endure all the hardships, submit to all the losses, keep the flag flying, hold the ports open!" Just so long as the working classes take that position, all our talk will be idle. For one, I am willing to leave the question there. I believe that the laboring people of this country are considering it. There are signs that they are giving it thoughtful attention. They have already made some utterances regarding it. I am willing to leave it to them. Those who are nearest in condition and circumstances to these people who are coming to our shores—I am willing to leave it to them whether, for the sake of the American standard of living, and the American rate of wages, the doors shall be at least temporarily closed.

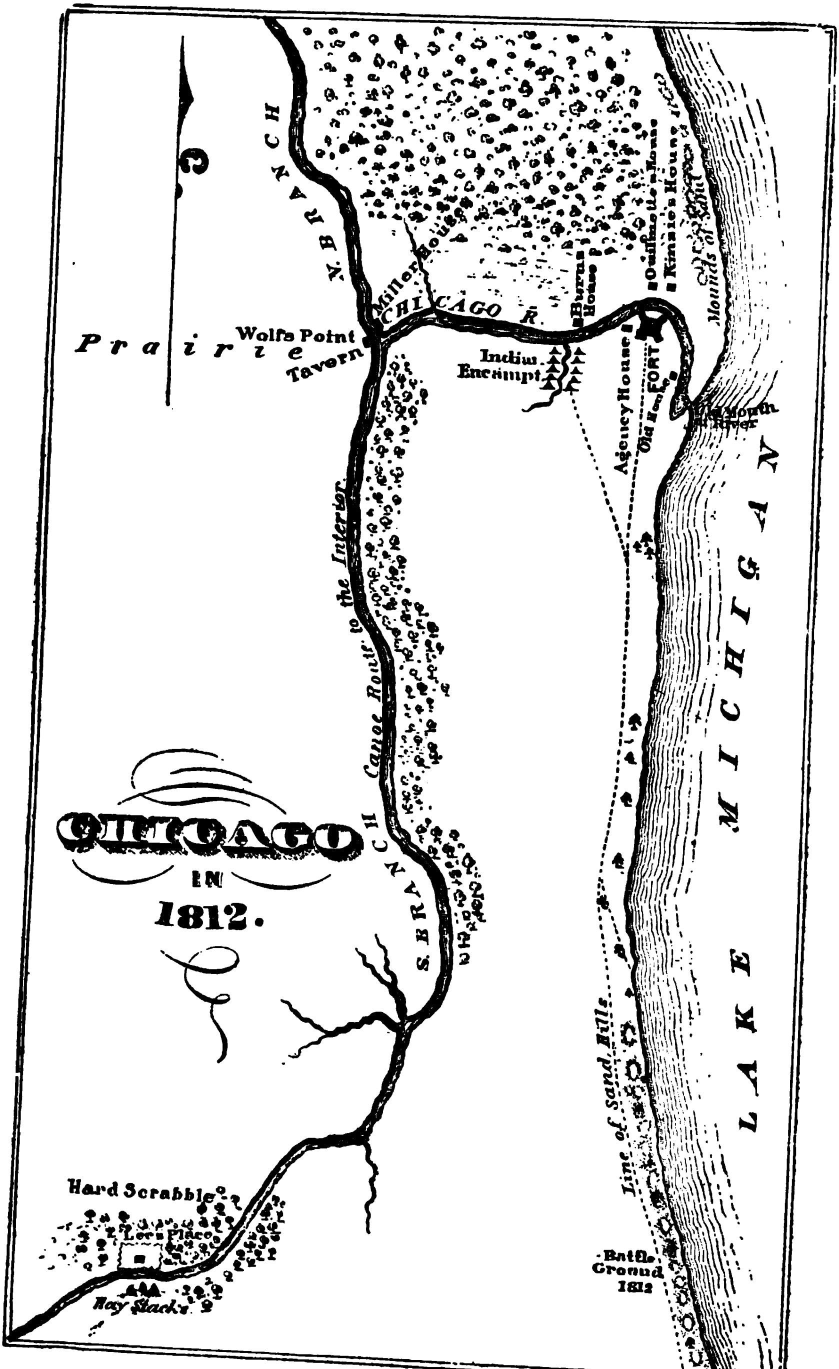

CHICAGO
IN
1812.
Prairie
Wolfs Point
Tavern
N. BRANCH
CHICAGO R.
Miller House
Burns House
Ouilmette House
Kinzie's House
Mounds of Sand
Indian Encampt.
Agency House
FORT
Old Mouth River
LAKE MICHIGAN
Canoe Route to the Interior
S. BRANCH
Line of Sand Hills
Battle Ground 1812
Hard Scrabble
Lee's Place
Hay Stacks

PLATE I.

The Sanitary District of Chicago.

By ISHAM RANDOLPH.

Past President Western Society of Engineers, Chief Engineer of the Sanitary District of Chicago.

When you call to mind the proverbial diffidence of the Chicago man, you will feel that it is meet that I should stand abashed in the presence of these learned professors and this assembly of their worthy disciples. But it is not of Chicago men and their diffidence that I propose to speak to you now, but of an enterprise which is worthy of the second city in population and the first city in area, ambition and achievement, which has sunken its foundations deep into American soil and reared its towers and palaces high into the free airs which blow where'er they list over the broad land whose states and territorties are bonded into the great American Republic.

We are too apt to assume that every body knows all about Chicago, and perhaps I ought to assume it now, but I believe that a brief statement of local conditions is a proper preface to a description of the great work with which I have the honor of being connected in a position of responsibility. You all know that Chicago lies near the southern extremity of Lake Michigan on its westerly side, latitude 41° 50′, longitude 87° 40′ nearly; but I hardly think that all of my auditors know that its plane of reference or the datum to which all of its levels are referred is 579.61 feet above mean tide at Sandy Hook. This datum was established in 1847 and was the low level of Lake Michigan for that year. The averge mean level of that Lake for a period of thirty-eight years has been about 1.74 above datum, or 581.35 above sea level. The fluctuations in lake levels, however, are considerable, the records for one year showing a minimum of —1.08 and a maximum of 3.66 or 4.74 feet between extremes.

A glance at the map, (Plate I) made in 1812 of the area which was destined to fill so large a place in American history, discloses nothing which could make destiny manifest; and the enthusiast who predicted on the 4th of July, 1836, at the exercises attendant upon the inaugura-

tion of work upon the Illinois and Michigan Canals, that within one hundred years Chicago would have a population of one hundred thousand souls, excited the scorn and derision of his hearers. If that man's knowledge of mundane affairs takes in the present development of the city, how his disembodied spirit must exult over those immortals about him whose bony fingers, dipped in the scorn of that day, have crumbled to dust; and if a spirit utters the language of earth we know that he has many times said to his associates on the other side, "What did I tell you?" But he was a bold man to stand there on his nail keg, or whatever platform there was beneath him, and with his gaze upon those wide spreading marshes, populated by frogs, musk-rats and water snakes, predict that a trading village whose few scattered houses must have stood upon stilts to keep them out of the wet, would become a city of any note.

Eleven years after that date, the city had a population of 16,000. But the completion of the Illinois & Michigan Canal in 1848 gave it an impetus so great that within seven years the population had outgrown wells as a source of water supply, and surface ditches or wooden boxes laid nearly at the lake level as a means of sewage disposal. In 1855 E. S. Chesbrough was elected City Engineer, and entered on that career of splendid service which has made his name illustrious. He devised a sewer system and inaugurated the elevation of street grades which was a condition precedent to the laying of sewers in the Chicago of that day; for the natural surface was so little above the lake level that there was no chance to secure even the slight grades which were adopted – three-fourths of an inch to 100 feet, or even less than that in some extreme cases,—nor to bury the sewers out of sight and prevent them from obstructing the streets which they occupied. To flush these sewers he mounted water tanks with large discharge valves on wheels, and at stated periods discharged the contents into the sewers. So successful has this method been, and so economical withal, that it is in vogue to-day, and one of the common street sights is a rectangular tank holding about sixty barrels of water and drawn by four stout horses, standing over a man-hole and delivering a torrent of pure water into the sewer beneath.

To meet the demand for pure water Mr. Chesbrough constructed the first tunnel under the lake, going out two miles into thirty feet of water for his in-take crib. This tunnel is 5 feet in horizontal diameter and 5 feet 2 inches in vertical diameter. To-day we have five tunnels extending out into the lake as follows: The Lake View Tunnel, 6 feet diameter, 2 miles long; the Chicago Avenue Tunnel, 7 feet diameter,

2 miles long; the Chicago Avenue Tunnel, (the Chesbrough) 5 feet diameter, 2 miles long; the Four Mile Tunnel, 8 feet diameter; and the 68th Street Tunnel, 6 feet diameter, 2 miles long. These tunnels are capable of supplying 434,000,000 gallons to the pumps daily, and the pumps now installed have a capacity of 357,000,000 gallons in 24 hours. This supply is distributed to the city through 1473 miles of pipe, of diameter varying from 3 inches to 48 inches. The total cost of the Chicago water system since 1861 has been $21,795,517.43. The sewer system which serves the city aggregates, 1,144 miles in length, of diameters ranging from 6 inches to 12 feet, built at a cost of $15,370,-944.79. These statistics were taken from the reports for 1893.

With the increase in population and material resources, filth increased prodigiously, producing results which were revolting to the nose and eye and menacing to the health of the great community. The Chicago river became a great open sewer, whose vile contents in time of freshet were forced into the lake, carrying pollution far out into that vast reservoir which nature made so pure and wholesome but which man has done so much to poison. Currents often sweep this unwholesome solution to the very in-takes through which the city draws its drinking water, and it is no uncommon thing for the Health Department to warn the people against the use of city water for drinking purposes, during the continuance of these freshets. The evils of sewage pollution had reached such a height in the early sixties that the city sought relief in the Illinois and Michigan Canal, and in 1865 entered upon the work of deepening that channel for the purpose of creating a gravity flow through the river toward the west, which should protect the lake in a measure and minimize the vileness of the river. This work was not completed until 1871, when the great fire made a vast crematory of the city. Then came the reconstruction period with its absorbing interests and its sore hardships. During this period much benefit was derived from the flow through the canal, as deepened; but with the accelerated growth in population and multiplied industries came a great augmentation of filth, with all its attendant evils. As a measure of relief pumping works were established at Bridgeport and in 1884–5 they entered upon a service which has continued until now. These pumps, four in number, were built under a guaranty of 60,000 cubic feet per minute, but that discharge has not been maintained. For long periods their performance has fallen below 40,000 cubic feet per minute, and while, on the principle that every little helps, the good that they have done has been in evidence, yet their inadequacy was still more apparent. An estimate based upon careful analyses of the water pumped from the

Chicago River into the old canal, extended over long periods, showed that in 1890 eleven hundred tons of solid matter found its way through these pumps every 24 hours.

An organization of public spirited men, known as the Citizens' Association, starting about the year 1880, entered upon a career of useful agitation and public education on broad lines of municipal development, paying most particular attention to water supply and drainage. Through their efforts the "Drainage and Water Supply Commission" was brought into being January 27, 1886. The Chief Engineer of this Commission was Mr. Rudolph Hering; consulting engineers, Benezette Williams and Samuel G. Artingstall, with L. E. Cooley principal assistant engineer. The commission considered various alternative schemes for solving the problem committed to them. They reported to the mayor and city council in January, 1887, in favor of a Drainage Channel and waterway from the Chicago River to Lockport, as being the most feasible and effective solution to be accomplished within reasonable limits of cost. Their conclusions, based upon careful investigations, were adverse to land disposal, and equally so to discharging all of the city sewage into the lake at some point removed from the center of population and taking the water supply from a location far enough in the opposite direction to preclude any possible contamination from sewage.

In 1889 the Sanitary District Law was proposed and under its provision the Sanitary District of Chicago was organized. The first election of Trustees was in November, 1889. They are elected by the legal voters residing in the District, and form the most independent civil corporation of which I have any knowledge. The life of each Board of Trustees is five years, and during that period these nine men are answerable to no other authority than the law which created them. They elect a president from among themselves whose term is one year, but he is eligible to re-election (the present president is serving his fourth year), a clerk, chief engineer, and attorney. They may levy and collect taxes for carrying on the work intrusted to them, to the extent of one-half of one per cent. per annum of the value of the taxable property within the corporate limits of the district, as the same shall be assessed and equalized for state and county taxes of the year in which the levy is made. They may issue bonds to the extent of five per cent. of the value of the taxable property of the district, as determined by the last assessment for state and county taxes previous to the issue of said bonds; provided, however, that said five per cent. shall not exceed the sum of fifteen million dollars.

PLATE II.--MAP SHOWING DIVIDE BETWEEN LAKE MICHIGAN AND MISSISSIPPI RIVER.

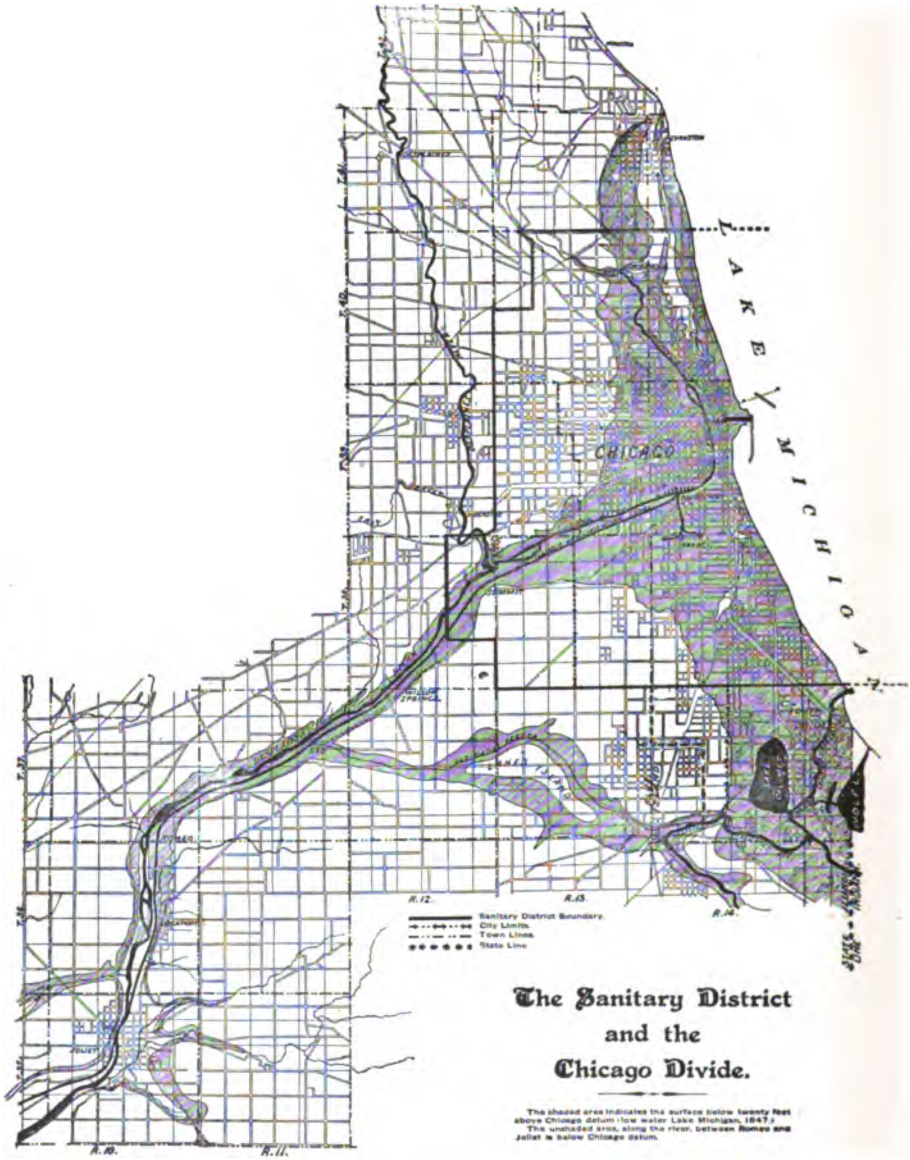


PLATE III.—LOCATION OF DRAINAGE CHANNEL IN THE DELPLAINES RIVER VALLEY.

Mr. L. E. Cooley was the first Chief Engineer of the district, a most proper recognition of his eminent ability and years of earnest work in preparing the way for the great enterprise. But his term of office was brief; there were elements at work which accomplished his removal before he had served twelve months. But the rank and file of the American people believe in fair play, and the voters rallied to his standard and he was elected to fill a vacancy made by the resignation of one of the trustees first elected, and the engineer whom they tried to discredit took his place upon the board as the peer of any there.

It is about time now that I should get down to the work of the channel itself. The Sanitary Law required that the channel should "be constructed of sufficient size and capacity to produce and maintain at all times a continuous flow of not less than 300,000 cubic feet of water per minute," "and that will produce and maintain at all times a continuous flow of not less than 20,000 cubic feet of water per minute for each 100,000 inhabitants."

The location of the channel was the occasion of much contention in the board. In prehistoric times the forces of nature had scooped a valley from Lake Michigan to the great valley of the Mississippi (see Plate II), and no doubt can be entertained of the fact that a majestic stream once flowed from the great lake basin, through this waterway, to the Gulf. But there came a subsidence of the waters in the great basin, until the shallows became a bar and the westward flow from its reservoir ceased entirely, leaving the furrow which was turned by the mighty shares of the glaciers to carry only the modest flow of the Desplaines water shed. The narrowness of the divide between these two basins was tersely described by Mr. L. E. Cooley, when he said, with almost literal truth, before the Deep Waterways Convention at Toronto, "Within the environs of Chicago a rain drop could split upon a blade of grass and part flow each way." The divide between these basins is about 9 feet above the level of Lake Michigan. It needed no engineering education to determine that the Sanitary Channel must follow the route of the glaciers, but there was a chance for the engineers to split something more tangible than hairs on the exact line to be followed down the valley. Those quarrels subsided and the channel was located as shown upon the map. (See Plate III.) The maximum curvature used is 36′.

Then came the hydraulic questions of cross section and slope proper for meeting the spirit and letter of the law, and happily a full investigation demonstrated that economy lay in the direction of a deep, rather than a wide channel, because the amount of material to be excavated above the

flow line, which adds nothing to channel capacity, is minimized in the deep channel. Thus economical construction dictated a waterway which met the fullest requirements of the sanitary condition, and at the same time afforded an ideal navigable channel.

A very exhaustive set of computation for channels of various dimensions was made, to enable the trustees to reach intelligent conclusions in regard to the proper cross section to be adopted. These computations were made by Mr. T. T. Johnston, Assistant Chief Engineer, aided by a corps of competent assistants. Of course, coming in as I did as the fifth Chief Engineer of the enterprise, I wanted to know how these conclusions were reached, and for my information Mr. Johnston prepared a very full report on the reasons which led up to his final conclusions. These reasons he epitomized for me in a statement which I cannot do better than report here in about his own language:

"The methods of computing the dimensions of the Main Channel and the River Diversion as well, are necessarily tentative in nature. All reliable formulæ for computing physical conditions must, of necessity, be based on experience with the dimensions and nature of the conditions to be computed. Such experience in water channels of large dimensions is quite limited, and none of the ordinary formulæ for calculating flow of water apply to conditions such as are met by the Sanitary District. This was realized early in the study of the drainage question in 1886–7. At that time a collection was made of all available experiments in the flow of water in open channels. The measured elements in such experiments are 'hydraulic radius,' 'slope' and 'velocity,' and any method of correlating these elements that will reduce their variation with one another to law and order serves as the basis for a formula by which any one of them can be computed when the other two are known. An arbitrary method (See Fig. 1) was adopted. Log. r was assumed for one axis, and (log. V— ½ log. S) for the other. The measured elements of all experiments were plotted to these axes with favorable results; a straight line such as A B was selected to represent the variations involved. Its equation reduced to numbers was $V = 22.39\, r\, S^{\frac{1}{2}}$. This formula, entirely empirical in its nature, of necessity has but limited application. It served for the work, however, and gives results esentially the same as Kutter's formula with "n = 0.030." The cross section of the earth channel for a flow of 600,000 cubic feet per minute with a depth of 22 feet was fixed at 202 feet on the bottom, side slopes 2 feet horizontal to 1 foot vertical, so that the width at water line becomes 290 feet. The grade (See Plate IV) is

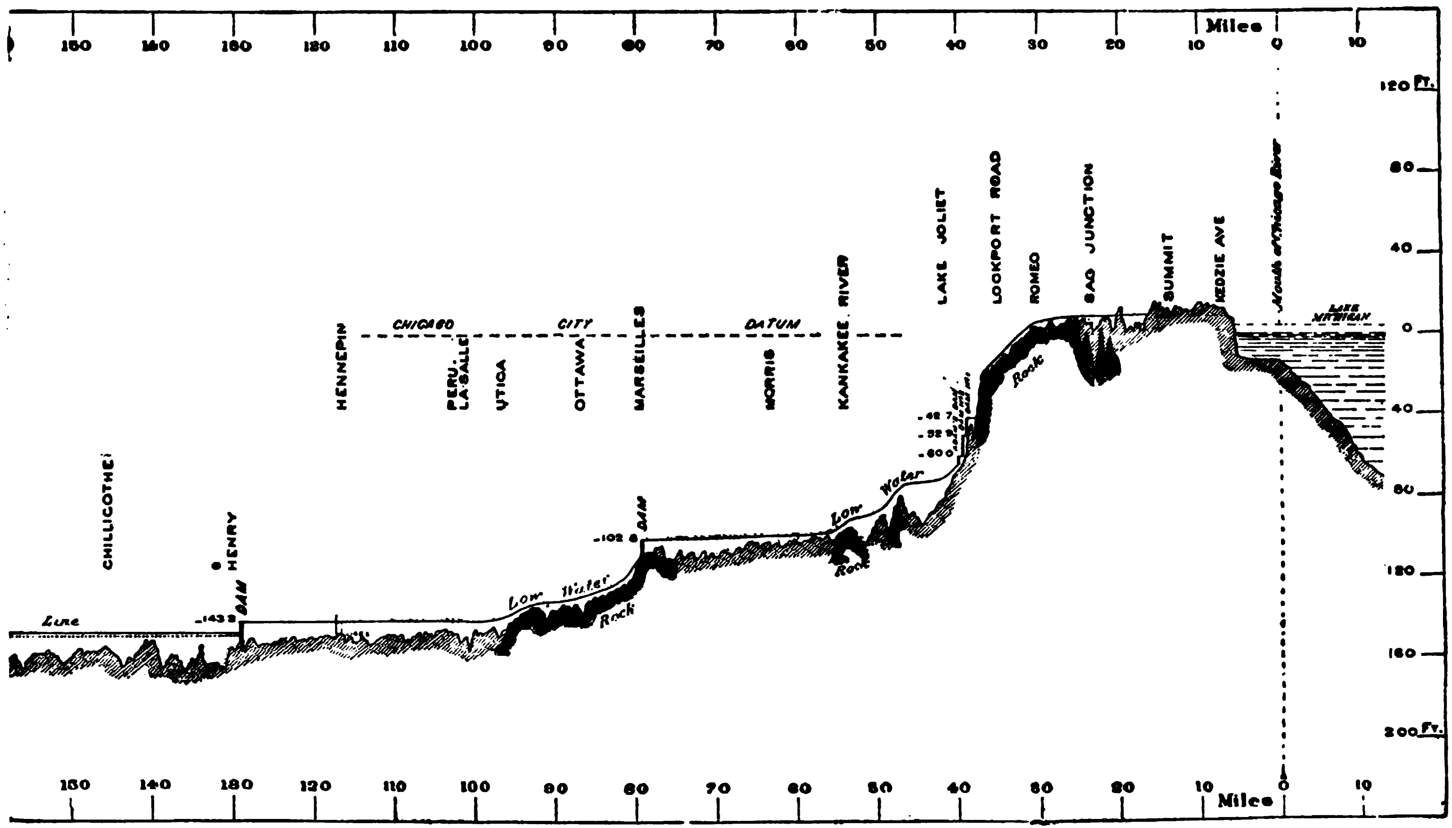


PLATE IV.—PROFILE OF DRAINAGE CHANNEL.

fixed at one foot in 40,000 feet, or 1⅝ inches per mile, nearly. Throughout the solid rock portion of the channel the cross section for the same flow is 160 feet, sides vertical, taken out in three stopes or levels with six inch off sets on each side for the first and second stopes, so

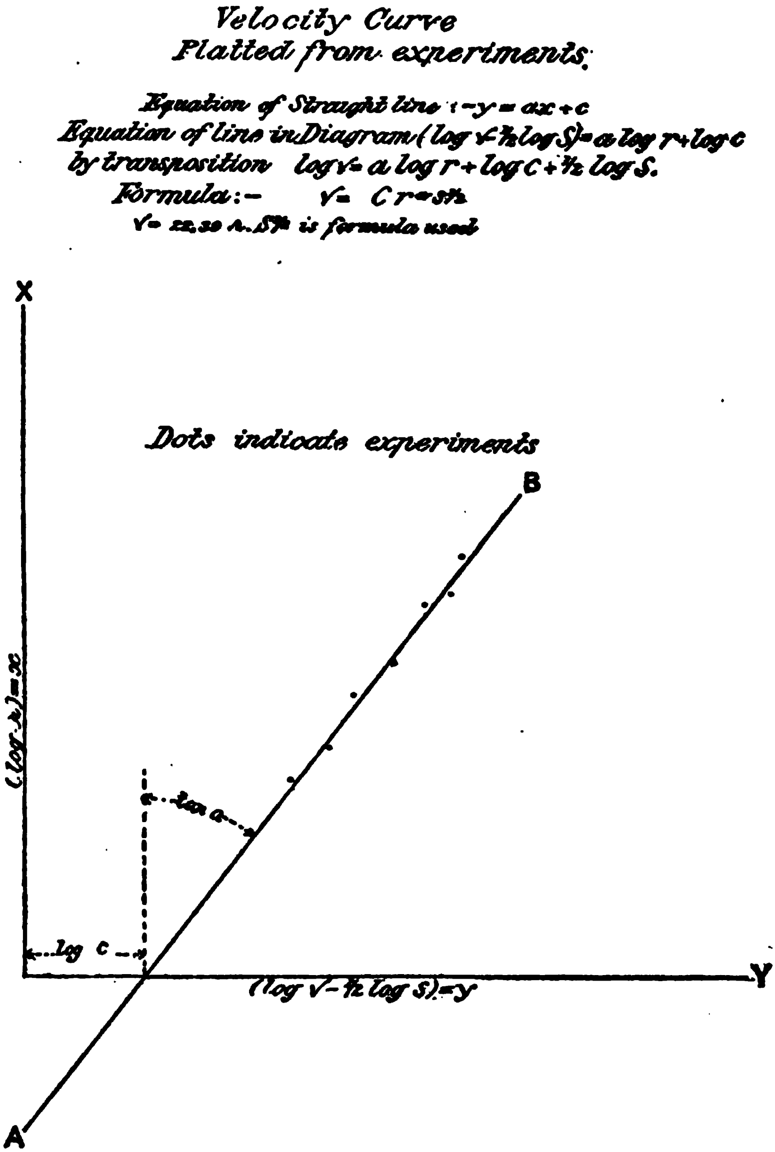


FIG. 1.

that the width at top is 162 feet. The grade in rock is one foot in 20,000, or 3¼ inches per mile. We had a chance on our River Diversion to see how nearly the theory of the Sanitary District formula checks out with the practice of the waters in flood times. In March, 1894, we had a flood, and of course our men were on hand to make all of the necessary measurements for determining, volume, velocity and slope, and the results bear out the theory to a most gratifying extent.

The accompanying charts show the comparative sizes of various great channels. (Pages 94, 95, 96.) The channel of the Chicago Sanitary District has the largest cross section of any of them.

The first thing that had to be done after the letting of the contracts for this work was to change the course of the Desplaines River. It came down the valley, meandering from side to side, in and out, sometimes in flood seasons covering the whole valley. It was of course nec-

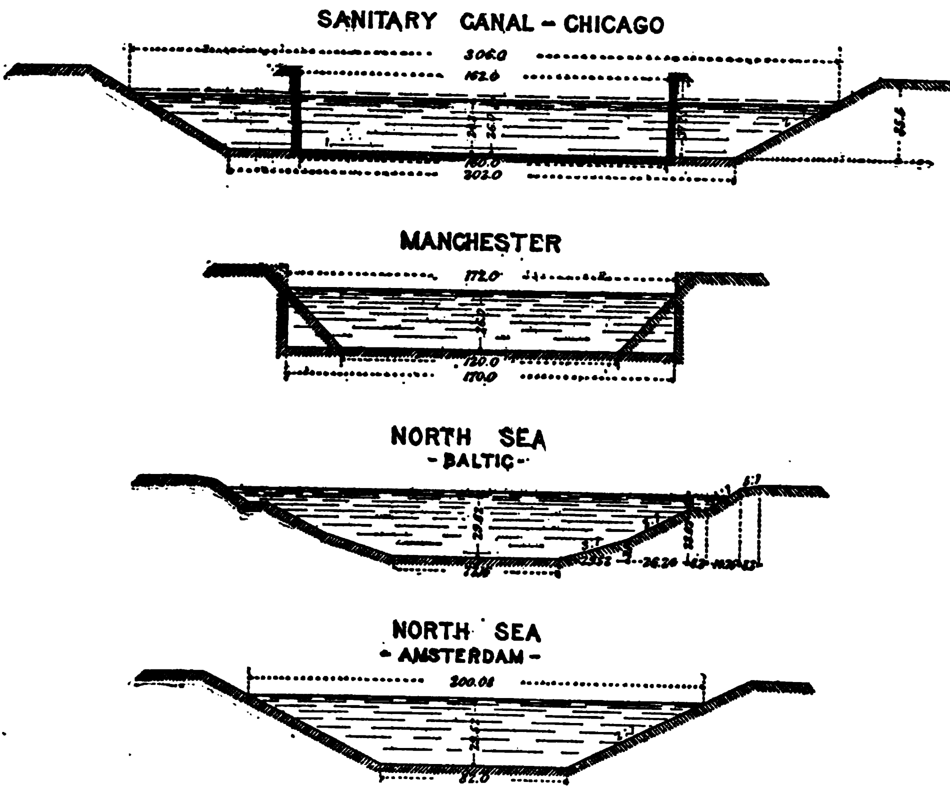


FIG. 2.—CROSS SECTIONS OF NOTED CHANNELS.

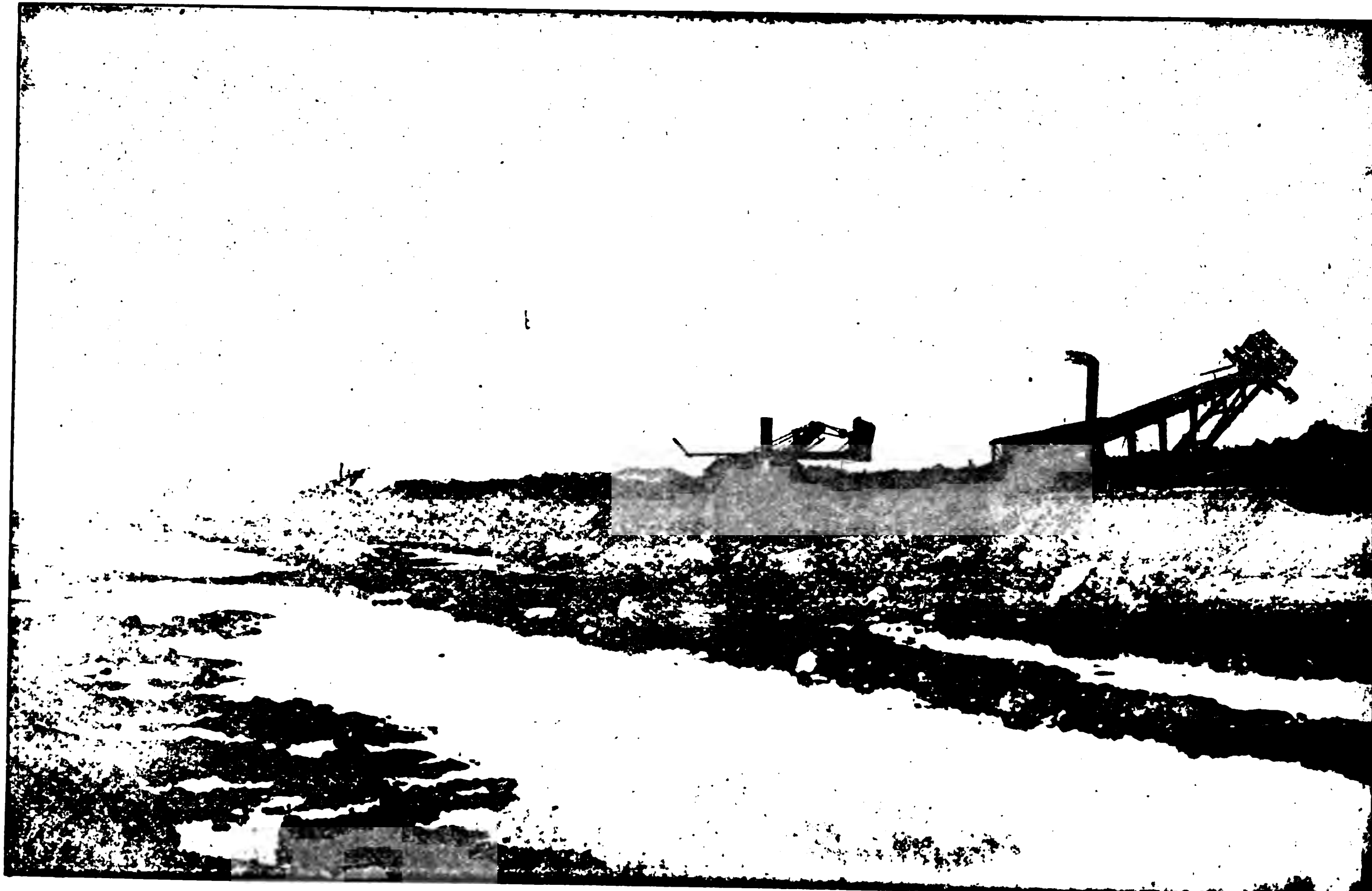


PLATE V.—NEW CHANNEL FOR DIVERSION OF THE DESPLAINES RIVER.

essary that that river should be turned out of its bed in order that we should do the work. It was necessary to make thirteen miles of new river channel and nineteen miles of levee. (See Plate V.) This was accomplished in a single season, at a cost of $1,060,000 in round numbers.

After this diversion was done we could go at the work on the main channel. When I came into the service of the District, June 7, 1893, the amount of excavation done on the channel was 1,062,398 cubic yards. The estimate to the first of the present month (April, 1895.) show that we have completed 20,348,518 cubic yards, so that during twenty-two months there have been excavated 10,186,130 cubic yards. The table on Page 97 shows the total estimated quantities of material involved, the amount which has been accomplished up to the present time, together with the estimated cost and the amount so far expended. It will be observed that the money value of the work completed has reached more than half the amount involved in the contracts. The work will thus be on the down grade now.

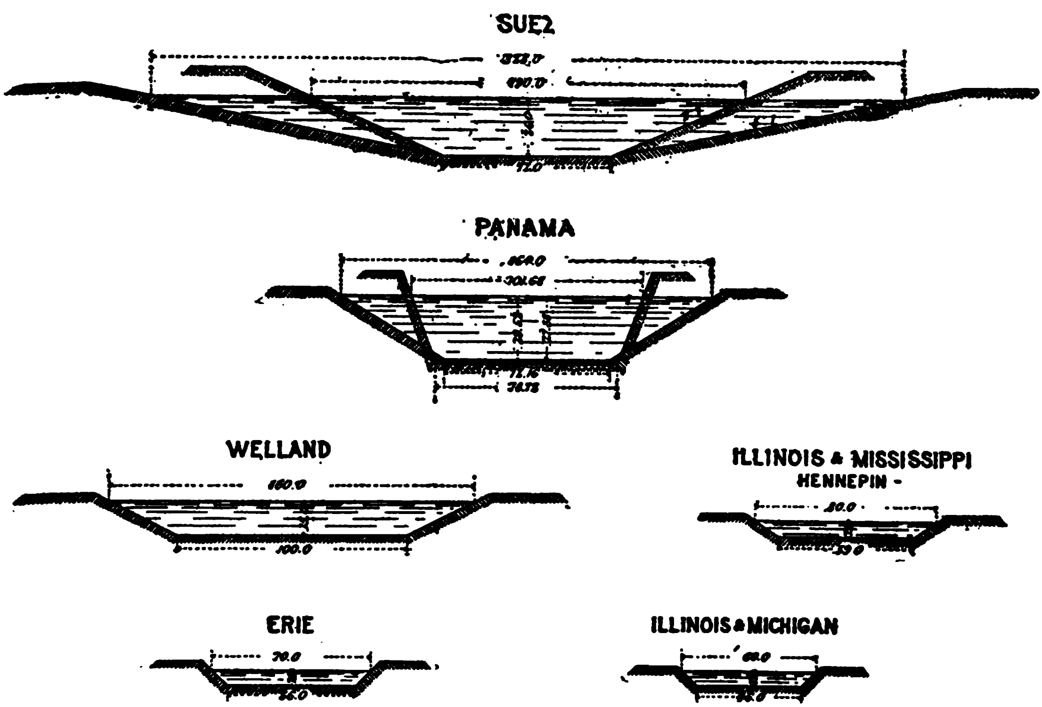


FIG. 3.—CROSS SECTIONS OF NOTED CHANNELS.

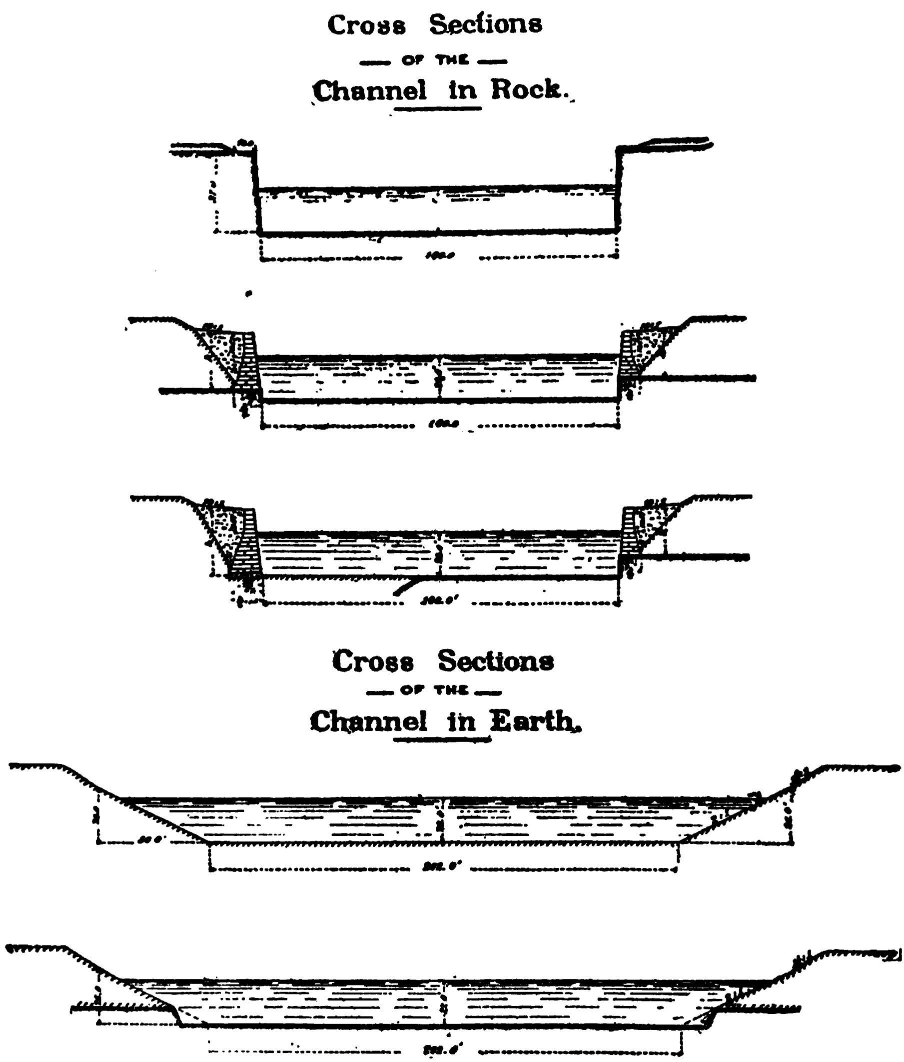


FIG. 4.—CROSS SECTIONS OF DRAINAGE CHANNEL AND WATERWAY.

PLATE VI.—CHARACTERISTIC EARTH CUT.—MAIN CHANNEL, SANITARY DISTRICT OF CHICAGO.—SECTIONS I AND K.

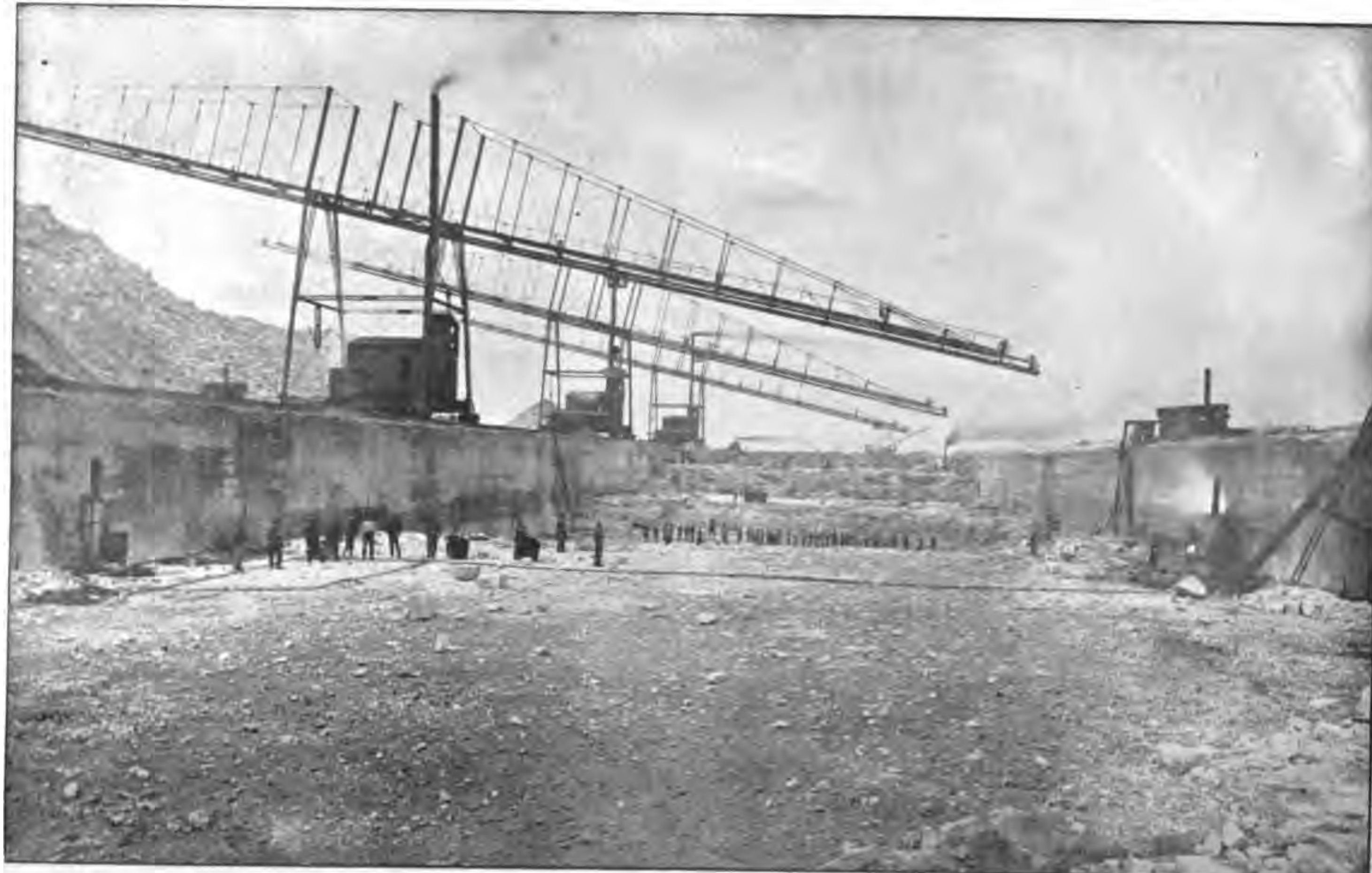


PLATE VII. EXCAVATING MACHINERY IN DRAINAGE CHANNEL. FULL DEPTH IN CENTER.

TABLE SHOWING VOLUMES OF MATERIAL INVOLVED AND AMOUNT COMPLETED APRIL 1, 1895, WITH STATEMENT OF COST.

Designation	Glacial Drift Cu. Yds.	Solid Rock Cu. Yds.	Ret'g. Walls Cu. Yds.	SlopePav'g. Sq. Yds.
Main Channel	26,077,765	12,071,668	384,958	1,285
River Diversion	1,564,403	258,926		
Total	27,642,168	12,330,594	384,958	1,285
Amount done April 1, 1895	13,816,484	6,532,034	2,600	
Balance required to complete	13,825,684	5,798,560	382,358	1,285

Total value of work under contract April 1, 1895		$18,990,078.50
Total amount paid or reserved under percentage, April 1, 1895	$9,555,729.67	
Total amount to be paid under present contracts	9,434,348.83	$18,990,078.50

The average cost of the work thus far has been 28.944 cents per cubic yard for glacial drift, and for solid rock 76.315 cents per cubic yard.

We are making at Robey Street, the Chicago end of the channel, a basin large enough to turn the largest vessels that ever entered the Chicago River. The water there at the present level of the lake will be 24½ feet deep. It may be stated here that the theory on which the depth of the channel was established was that if we have to bring the water through the Chicago River, there would be a slope of nearly two feet from the level of Lake Michigan to this point, so that we assumed here 22 ft. depth of water.

If moreover we can carry the water directly through the city, which may be done, we shall have really 24 feet of water throughout the channel.

One of the neatest methods of hoisting earth that we have in the channel is by the use of a bridge (See Plate VI) with its abutments mounted upon trucks which run on tracks parallel to the channel. The width this spans is 210 feet. The car is hauled up by hoisting machinery, and dumps anywhere within the length of that span of 210 feet. There are four of these bridges with their equipment of steam shovels and cars employed on Sections I and K.

The monster machine of the channel (See Plate VII) is one that spans it entirely while its cantilever arms extend out on each side so that it is 640 feet from tip to tip.

It was estimated that the work of that machine would be 5000 cubic yards in ten hours. But just as they got to work with it it was wrecked

by a wind storm. It is now being repaired. The original cost of this immense machine was about $32,000. I suppose we shall have to spend about $15,000 in rebuilding it.

In some cases a rubber belt is used for conveying away the material taken out by the shovel. The contractor undertook at first to load directly on to the belt with the steam shovel, but the masses of material blocked it so that it would not work. Then a pulverizer was gotten up. The steam shovel dumps its load into the pulverizer whence it falls upon the belt and is carried up the incline. The hoisting machinery is located in a car which is moved along parallel to the channel as the excavation proceeds.

A safety valve had to be built at the head of our River Diversion. The beginning of the nineteen-mile levee which we built is just above the bulkhead seen on Plate VIII. Before this wasdone, whenever a freshet of 50,000 cu. ft. a minute occurred in the Desplaines River, it began to flow toward Chicago. When we changed the course of the river, it was determined to do Chicago as much good as possible before the main channel was completed. So we sought to turn all of the flow of waters down the Desplaines through Joliet. Now nothing flows toward Chicago until the volume passing Riverside exceeds 300,000 cu. ft. a minute. This structure is of concrete, made of American cement, faced with Portland cement, and capped with cut stone, and the wing walls faced with cut stone. It is 397 feet in length, and the top of it is 16.25 feet above Chicago.

In March, 1894, the levee protecting Section E, broke and the section was flooded. The steam shovels at work there were covered so that only the tops of the booms could be seen sticking out of the water. The channel thus filled looked about as it will when the work is completed.

The New Era Grader is one of the most successful methods of excavating at the surface. The machine consists essentially of a plow which throws the earth on to a travelling apron. It is drawn by fourteen or sixteen horses or mules, and wagons driven along by the side receive the material from the apron and carry it away to the spoil bank. This is a very economical method of removing light soils.

One of the most successful machines used on the entire channel was on sections A and B. These lay for their entire distance in the old bed of the Desplaines River, and the muck overlying the solid material varied in depth from four to twenty feet. It was quite a conundrum to know how to get rid of that stuff. Finally, we got the owners of this hydraulic dredge into communication with the contractors. They

PLATE VIII.—DESPLAINES RIVER SPILLWAY, NORTH OF SUMMIT.

PLATE IX.—DRAINAGE CANAL AND WATERWAY IN ROCK. FULL WIDTH AND ONE THIRD REQUIRED DEPTH.

brought in one of their dredges and built another on the spot. The dredge consists essentially of a hull 105 feet long and 33 feet wide, on which a battery of four boilers is placed, and a centrifugal pump 6 feet in diameter run by a 250 horse power Westinghouse engine. The pump is keyed to the same shaft as the engine. There is a second smaller engine which operates the cutter at the end of the suction pipe. There are lines thrown out from one end of the dredge and made fast to each shore. By these the cutter is vibrated. It swings backwards and forwards across the stream cutting as it goes. The eroded material is sucked up by the pumps and delivered through the discharge pipe on the settling basin. The suction is 20″, discharge 18″. The capacity of one of these dredges as shown by its maximum performance on this work is 11,000 cu. yds. in 24 hours.

The building of the River Diversion was emergency work. It had to be put through in one season. One portion of it was particularly difficult, owing to the hardness of the material. At one time 2700 men were put at work in the distance of three-quarters of a mile, making a very animated scene.

Powerful steam shovels were used for excavating the glacial drift. They handle the difficult glacial drift very successfully. It is proper to state that we have only two classifications of material in our work: glacial drift and solid rock. Glacial drift, according to the specifications, comprises earth, clay, muck, sand, gravel, rock detached from its original bed, and any other material overlying the solid rock. Some of the contractors took the work at very low prices indeed, and found that glacial drift was a very different thing from what they had anticipated. They were forced to the wall, and the sections had to be re-let at higher prices. Plate IX represents a double hoisting machine for taking material out of the cut. The loaded cars are brought to the foot of the incline by horses, the cars are then drawn up by the cable and taken away by other horses to the dumping ground.

We are to have throughout those sections which are underlaid by rock, but are not through rock cuts, a wall above the rock. It will take 384,000 yds. to wall these sections. When I took charge of this work, I found that the specifications called for dry rubble walls, but I very soon made up my mind that no rock found in the sections was fit to build a dry rubble wall for such a work as this. After pleading the case for eight or nine months we got the Board to recognize that fact, and to change the specifications to cement rubble walls. The specifications require sand and cement in equal parts for the mortar. Native cement is used, the brands being chiefly Louisville and Utica. The first piece of wall was condemned. The contractor under-

took to disregard specifications and orders, with results so unsatisfactory that drastic measures had to be resorted to. He refused to tear down the work, and we refused to pay for it, and held back the money to pay for tearing it down. The wall stood there till this Spring, when he wanted to join on to it with this season's work. Finally, I had him tear down a part of the wall. One could take up handfuls of the mortar and rub it a little, and it would become sand. One of the laborers, when asked to hand up a piece of that mortar, said, "Hum, that ain't mortar, that's sand."

We have nine miles of the rough rock cutting, averaging 33 feet in depth. Before the better appliances were brought into the work, the contractor started in with carts, tram cars, etc. The ablest contractors we have are a firm of men who have done railroad work all over this country, from Maine to Texas. They have six sections to do. The head of the firm says, that from the hour when he took the contract, he could hardly sleep at night because it seemed impossible for them to come out whole at the prices fixed. But they put in a great deal of improved machinery, and now they are making money. (See Plate X.)

In one of the high-power derricks, the turn table is mounted on trucks. The boom is 106 feet long. It lifts a load, makes a complete revolution, dumps it and comes back. When the machine was first employed it was pronounced a failure, as there was no means of dumping the cars. They had to let down the skip or bucket and a man had to go and unhook the chains, and altogether it was so slow that the contractors could not work with it. One of the contractors, however, thought that he could devise a method for overcoming the difficulty, which he did by a very simple addition to the machinery. With this improvement the table way is a most efficient and admirable appliance.

One of the largest derricks has the booms 163 feet long. It is mounted on a turn-table 30 feet in diameter; while the skips are being emptied from one boom, loaded skips are being picked up by the other. This machine has a rotating gear just like a swing bridge and makes a complete revolution every time it unloads. When this machine was first built it was not a success. If it was worked fast enough to accomplish what it was required to do, the pinion was twisted off or the rest stripped. The base then was only 18 feet in diameter. But it was rebuilt with a 30-foot base and increased power in all the parts, so that now it works very successfully, indeed. Four machines of this type are used on one section of this work.

The finest device for handling rock in cuttings of this kind, of which we have any knowledge is the Brown cantilever, shown in the illus-

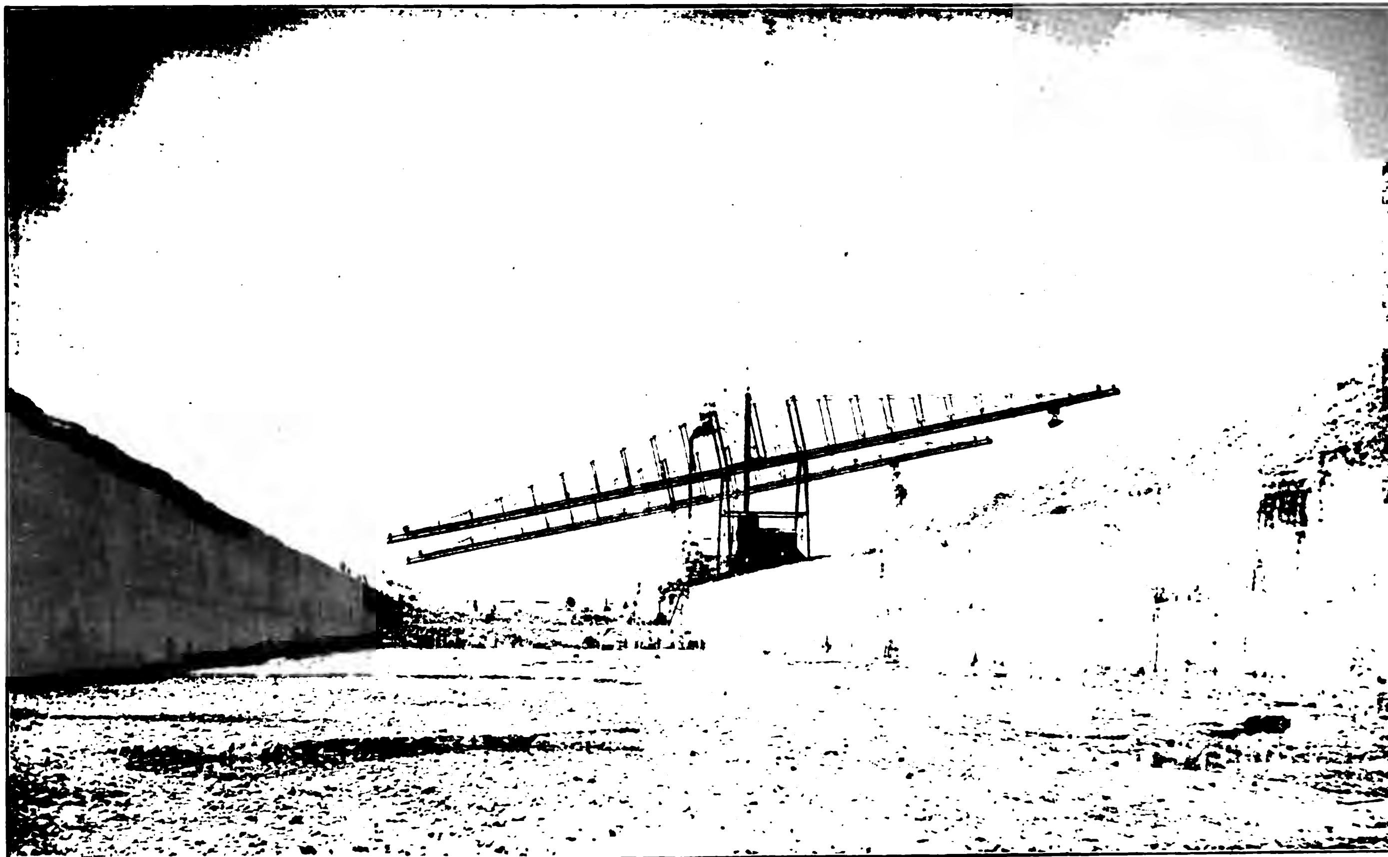


PLATE X.—CHARACTERISTIC ROCK CUT.—MAIN CHANNEL, SANITARY DISTRICT OF CHICAGO.—SECTION 10.

PLATE XI.—ROCK IN DRAINAGE CHANNEL AND WATERWAY AFTER BLASTING.

tration (Plate XI). We have no idea of what the capacity of this machine is, because we have never been able to get men enough on the face of the cut to load the skips fast enough for the machine to be kept steadily at work. The skip is hooked on, run up, and delivered in about fifty seconds. The weight of the car with its load is about 10,000 pounds.

The illustration shows the work in the bottom of the pit. There are three stopes with an offset for the first and second. About forty men are working on the face. In blasting the face, about twenty-four holes are drilled right across the channel a few feet back of the face. These are loaded according to the character of the rock, with from 500 to 800 pounds of dynamite, which makes a very pretty explosion. Our own photographer does not care to take his life in his hands photographing explosions, so keeps at a safe distance. The finest photograph of a blast I have ever seen, was taken by a photographer from Lemont, Ill. He went up within 300 feet of the face of an 800-pound dynamite blast. Then he waited for his shot until the rock loosened by the blast began to rise above the smoke, so that it appeared in the photograph coming down in showers. Ordinarily the photographs show only the cloud of smoke before the rock comes into sight.

We have two styles of channelling machines on the work. One is built by the Sullivan Channelling Machine Co., and the other by the Ingersoll Sargent Co. The machines consist essentially of a vertical boiler with engine attached, operating a great chisel which is worked by a steam piston, which is kept pounding up and down. They move backward and forward on a track about sixteen feet long, cutting a channel about ten feet long and ten feet deep in the course of a day. It adds, on a channel of these dimensions, about six cents a cubic yard to the cost of every yard taken out. The later Sullivan machines are improved by the addition of a steam dome which surrounds the smoke stack and give much better results. The cutting edge of the chisel is shaped like a letter *Z* and cuts a channel about three inches wide.

The drilling on this work is done almost entirely by compressed air. Each section using air has its own compressor plant, and it has proved a great saving in cost, while its convenience is a still greater feature in its favor.

Plate X shows a stretch of completed channel and by the 1st of December of this year there will be about seven miles of this rock work completed. The first blast was fired on this work September 3, 1892. If we can get the money, the work will be completed in the Fall of 1896. But unless the legislature in the next two or three days passes a law authorizing the raising of the necessary money, about

$9,000,000 will be lacking of what is necessary to finish it. By the first of next December about $18,000,000 will have been spent. The contractors on this work have invested in machinery on this channel about two and three-quarter millions of dollars. That investment will be practically sunk when the work is completed. The machinery will be little more than junk.

I think I have described all the features of the work except the controlling works. We have decided to put in there one section of a Bear trap dam, which is a moveable dam raised and lowered by the water itself. Besides there will be four sections of lifting gates of the Stonygate type. The reason for this is that the flow through this channel has to be controlled. At times there will be a volume of water passing down the Desplaines of at least 800,000 cu. ft. per minute. With the 600,000 cu. ft. per minute from the Sanitary Channel added to it, this water would be likely to do great damage in the city of Joliet. By means of these gates we can control the flow of the channel, so as to give a fluctuation of about thirteen feet in depth. At normal stages the water will be eight feet below datum. We shall have to hold it up as high as five feet above the ordinary stage of Lake Michigan in extreme cases. The people of Joliet and the Desplaines valley are trying to get a bill through the Legislature compelling the city of Chicago to extend this work as a navigable channel down the slope to Joliet. This would involve building two costly locks and would mean an expenditure of about $5,000,000 more, which would be solely for the benefit of Joliet.

If the necessary money could be obtained to develop it, a water power of 40,000 horse-power could be obtained at the end of this channel. All that can be done now is to buy the land for future development, and wait for the coming of the money, which will probably be in a few years.

There are now about 110 men in the engineer corps of the District. The channel is being worked in four Divisions, each in charge of an assistant engineer. The regular force for a division consists of an assistant engineer, sub-assistant engineer, 2 instrument men, 2 sub-instrument men, 2 computers, 2 rod men, 6 chainmen and helpers.

In the draughting room ten men are employed. In the record division where all estimates are checked over and the records of the department are kept, eight men are employed. In the section of designs and tests there are from six to ten men employed. In the department of surveys there are eight men. The chief engineer is immediately assisted by an assistant chief engineer, a superintendent of construction, and a principal assistant engineer. With the opening up of retaining wall work an increase of five will be necessary as this work requires very vigilant inspection.

PLATE XII.—LOADING CARS WITH ROCK IN DRAINAGE CHANNEL.

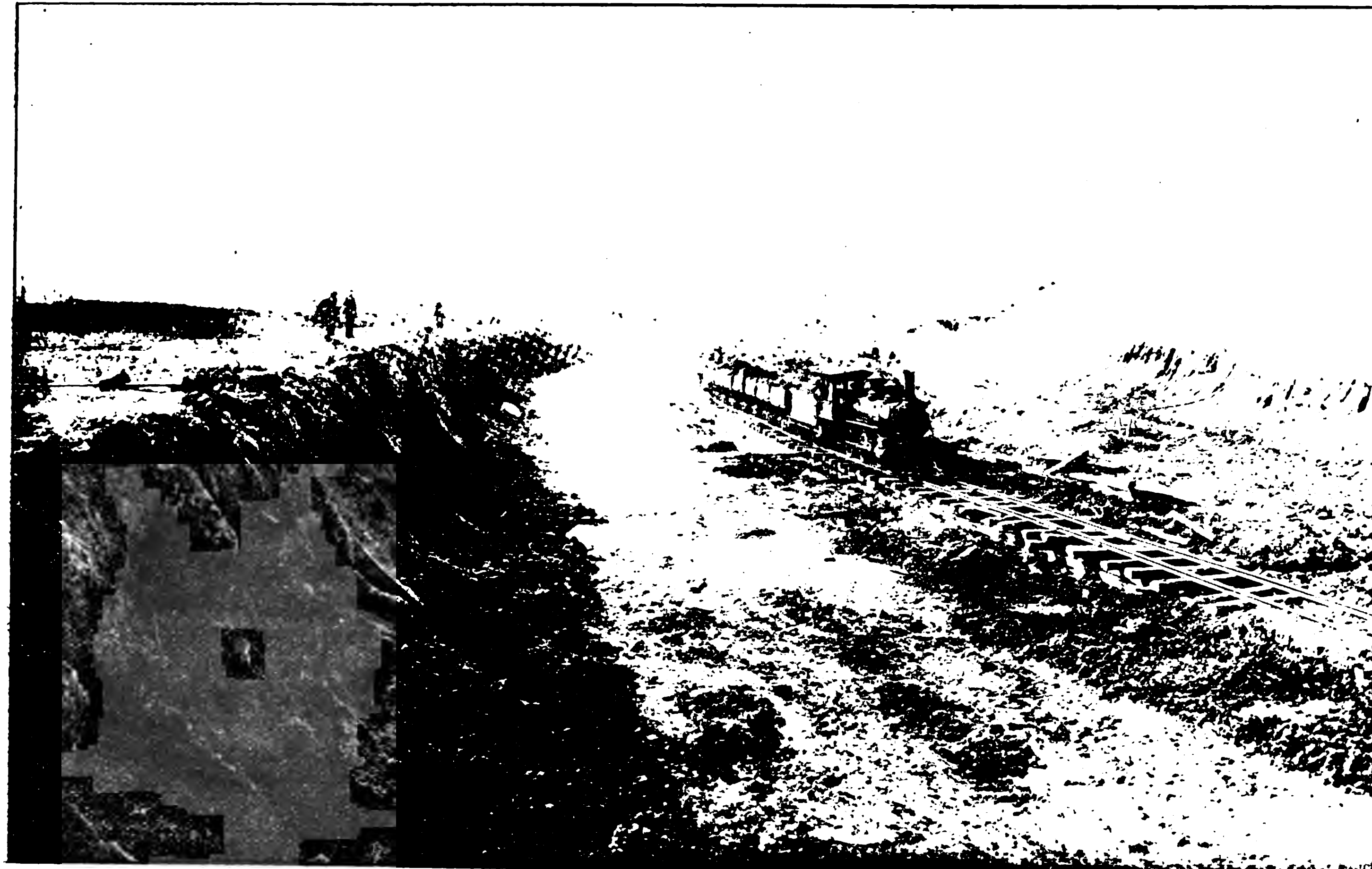


Plate XIII.—Excavation in Earth for Drainage Channel and Waterway. One-Third Required Depth.

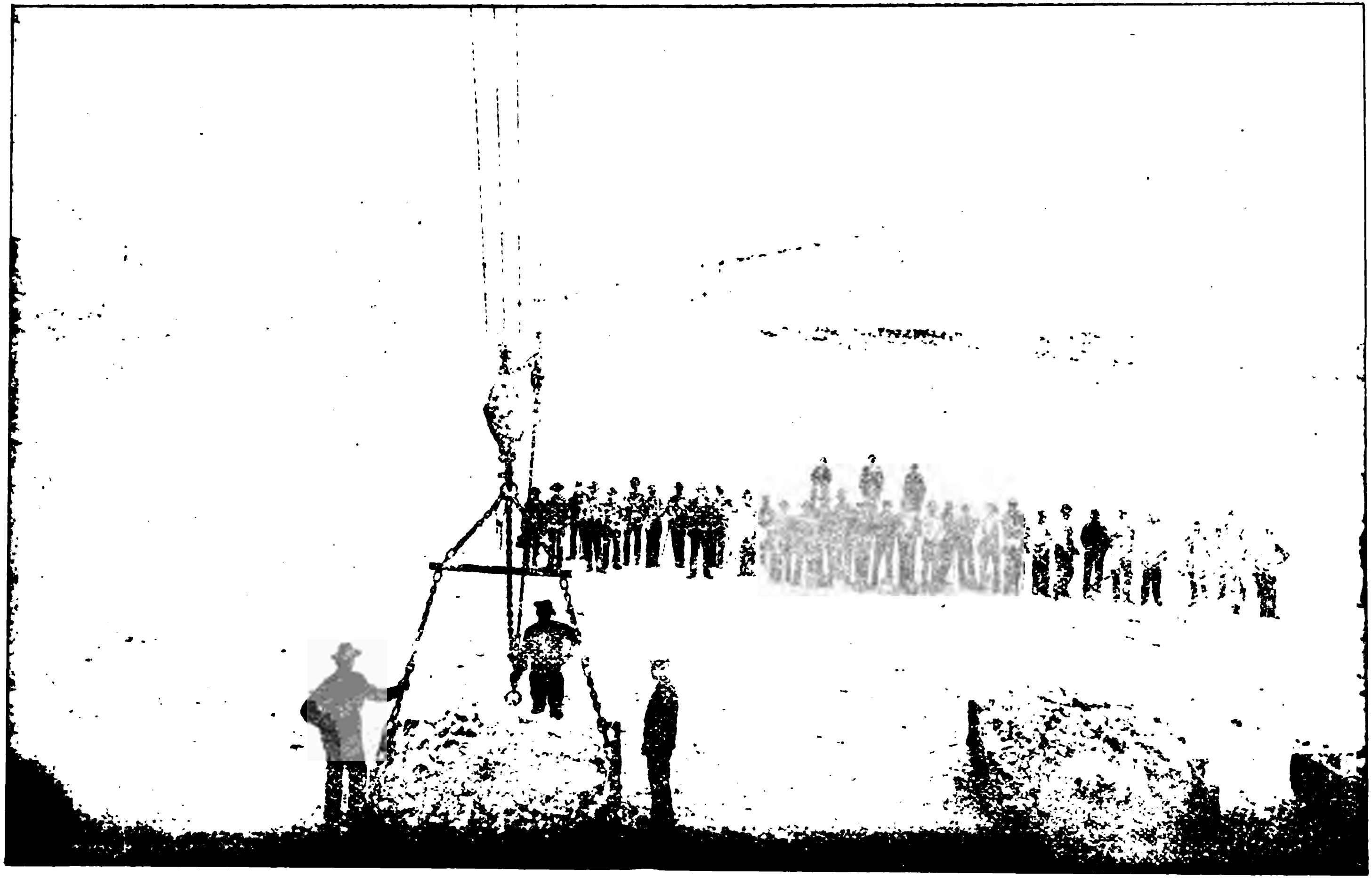


Plate XIV.—METHOD OF REMOVING ROCK FROM DRAINAGE CHANNEL AND WATERWAY.

The City and the Engineer.

ROBERT E. McMATH, M.AM.SOC.C.E.

President of the Board of Public Improvements, St. Louis, Mo.

The thing and person named in my subject are inseparably co-related. If there were no cities there would be no engineers, for there would be no need of any; and if no engineers were to be found, cities could not be built. You will see that I here use the terms in a broad sense. The city as the place of abode and work of human beings in considerable numbers; the engineer as he who moulds and puts to use the materials and forces furnished by nature. This last definition is too broad, if we think only of professional work, but I will let it stand with the reservation that I do not refer, in what I may have to say in this discourse, to the part borne by the merely skilled workman.

Speaking to young men it is appropriate that the topic and its treatment should be of the nature of a forward look rather than backward, for it is more profitable for you to have hints as to the character of the responsibilities you need to fit yourselves for, rather than tell you of what those who have gone before have done. Deeds which you will never be called on to repeat. None of those who have wrought in the field of city engineering have achieved undying fame. We do not know who was city engineer of Babylon in the days of Nebuchadnezzar, nor of Rome in the days of its glory, but we do know that they were competent men, for things were done in their days that would tax the abilities of the best of modern engineers. We know, moreover, that engineers were not, even in remote times, all from the ranks of practical men in the sense that term is often used as in contrast to scientific. Archimedes was an engineer of technical education, who put his theoretical knowledge to practical use. King Solomon, with all his wisdom, was wise enough to send to Tyre for an architect engineer, who studied out his plans, before he began work, so thoroughly that the hammer of the fitter was not heard about the building.

When notable engineering achievements are mentioned our thoughts tend towards rivers and gorges bridged, mountains tunnelled, great

lines of transportation brought into existence, railways and canals located and constructed. These certainly are grand undertakings—but they are not now, and in days to come will still less be the occupation of the great body of those who follow the profession of the Engineer.

My object will be to set before you, as young men looking to engineering as your probable life work, the fact, as I regard it, that engineering in and for cities is an inviting field for you to look over and perhaps enter of set purpose to find there your work.

I am aware that some regard a nomadic life for some years as a valuable part of a civil engineer's education, and I do not wish to gainsay any argument that has been advanced,—but the nomadic feature has an undue attraction to many young men—and in my judgment, so far as roughing it enters into the life, it brings nothing of value. A young man will be the better for a broad experience gained by serving under many chiefs and in diverse lines of work, so as to become familiar with a variety of methods before forming fixed notions and habits, and also to shake off any tendency to follow in the steps of a master, and further, if you will excuse me, to get some of the conceit taken out of him. Beyond this I know of no reason why he should not locate permanently at an early date. In my opinion, he will, as a rule, be the gainer if he selects his place of abode and never changes his residence; going from it as occasion offers, but returning to it. "A rolling stone," by the proverb, "gathers no moss." An engineer should not gather moss on his back, neither should he wear himself out by rolling. There is a happy mean that is worth seeking.

I will fail of my object if you think my suggestions have sole or even principal reference to official positions. Cities furnish many opportunities for Municipal engineering outside of direct employment by the City, and, so far as my observation goes, when a Chief is to be appointed, the fortunate party is usually one who has wrought in these outside lines rather than one taken from the ranks already in city employ. Under a well administered Civil Service system, this might not be the case.

To forecast the future of Municipal engineering, we must first ascertain what the cities of coming days are to be. There is a proper use of the imagination, which is essential to success. One must first conceive the idea, then picture in his mind the object in outline, and afterward perfect the conception by working it out in appropriate details. In my own life, I have gone through some preliminary stages and have dismissed as too visionary for serious consideration projects which I have seen conceived, taken up and successfully carried out by others, who had more faith in their imagination than I, or better judgment.

Poets have sung of the excellencies of pastoral life and spoken disparagingly of cities. "God made the country, and man made the city," says Cowper, and in the same strain Cowley voices the reproach: "God the first garden made, and the first city, Cain." I am not aware that these writers give evidence that they ever read the Good Book through, or they would have at least qualified their implied condemnation of the city, for He who at first put man into a garden to meet and yield to temptation, in the end intends that perfected man shall have his permanent home in a City. A study of the specifications of that City shows that it is to be beautifully adorned, perfect in sanitary arrangements, well paved and the pavements kept clean, without slum districts or dark places, abundantly supplied with pure water, food so ample and secure in supply and distribution, that none of the inhabitants can lack.

We speak of cities as well-built, and call some of our modern style buildings—"sky-scrapers." But Chicago or New York's tallest are but dwarfs compared with the lofty architecture that shall be characteristic of the final city, and the beauty of its buildings will appear from whatever point or angle viewed,—not as ours, fair in front, but hideous from side, rear or distant view.

I have named the points in the specifications which pertain to engineering, and would impress you with the thought that municipal engineering has a future, without limit in time and scarcely in scope. The Chief Engineer has dropped a few hints as to the ultimate plan, and has left to us—his assistants—the privilege of filling in the outlines and working out the details. It is as much a privilege and honor to lay foundations as to gild fineals.

Coming to the realm of present facts, as demonstrated by the actions of men, man is made for life in communities, and the community becomes by natural development a city. Cities have been a chief factor in civilization—"redemption from savagery." Cities furnish the opportunity for men to make the further progressive step to enlightenment, coming under the perfected law of liberty, by which each man's free acts will not infringe upon the right of any other. Perfect enlightenment is the realization of an ideal social state to which close and necessary association is an essential condition. The city has throughout the history of the race been the center of activities, the initial point of action.

In the centuries preceding the nineteenth, the size of the cities was limited by ability to command supplies; a maritime location alone enabled a great city to be fed. Steam and the railway have in great

measure done away with the limitation. Men may now gather to any number, anywhere that water in sufficient quantity can be had, and bear in mind that the quantity of this necessity per capita increases with progress in civilization. Moreover, density of population finds a limit in sanitary conditions. With steam, railways, electricity, telegraph and telephone, have come improved implements and machinery in all lines of industry, whereby the labor of one man in field and factory accomplishes what formerly required a dozen. Hence production of food is no longer the necessary avocation of a majority of men, and laborers are made free or even compelled to find other employment than agricultural. Machinery has also rendered the continuance of many home industries impossible, and the manufactory has displaced small shops. These changes are the result of development and have come to stay, and men are and will increasingly be compelled to congregate in large masses in order to win a living.

The growth of cities in the old world and new alike has been astonishing for the century now drawing to an end, and especially for the last half of it. Even in states whose history scarce covers fifty years, the country parts and small towns are actually decreasing in population, while the growth of cities, and larger towns on the way to become cities, shows a high rate of increase. So far from being disturbed by the facts and bemoaning their existence as indicative of evil, we must recognize, if not welcome, them and prepare to meet the coming situation.

Such preparation, in the first instance, calls for a high degree of practical, large-minded wisdom on the part of those whose province it is to legislate concerning municipal affairs and to administer them, and I may add on the part of courts when interpreting law, for new conditions will require the modification of many positions taken by courts in restraint of powers needed for great cities.

It might be interesting to inquire why this drift of population to cities has developed and progressed so rapidly, but our time and my purpose do not permit going into detail or proofs. I content myself with stating these propositions:

1. The movement is in accord with the social instincts of the people of the latter part of the nineteenth century, and is a result of universal education and the printing press.

2. External conditions now permit these great aggregations, as not inconsistent with necessities, comforts and conveniences of life.

3. External conditions not only permit, but exert a certain amount of compulsion for many to leave the country and seek the city.

These external conditions are largely the result of inventions and discoveries in the useful arts and sciences.

These impelling forces drive those who have a living or fortune to make or mar, cityward, regardless of the probable outcome of good or evil. In my opinion the balance of probability is in favor of good as a whole, notwithstanding the fact that it is easier to slide down than to climb up, in a moral sense as well as in a material. But I acknowledge, that unless wisely directed effort is put forth to provide the city populations with healthful influences, there is much of danger to country and race from the movement of the masses towards cities.

An economical theory that is much urged in these days charges that a large part of the ills affecting the masses is due to the individual ownership of land; claiming that poverty exists and increases because the masses are shut out from using the soil with the freedom that they have to light and air. To set aside these theories as untrue in essence and impracticable of application, it is enough that, as a fact, it is not possible to drive the average city bred poor man to the soil, and further, that he would starve in hopeless impotency to get anything from the soil if placed upon it without preparation and capital. If he could realize the delusion of the newly enfranchised negro of early post-bellum days, that with freedom he was to have "forty acres and a mule," he would be likely to desert the acres and make the mule carry him back to town, despite the familiar privations he must there undergo. "Better fifty years of Europe than a Cycle of Cathay," expresses a psychological principle deeply rooted in human lives.

To my notion, it is not access to the soil for cultivation that the masses need, but for a place to live and have a home. Better if each can own land, house, and all that goes to make home comfortable and attractive. Well, if each can have a separate house with grounds for children to exercise and play in without resort to the streets and their mixed associations. It is endurable if each family can have rooms and separate privileges, such as are afforded by the better class of apartment houses or flats; but we can go no lower in the scale of home conditions, and reasonably hope that parents and children will climb rather than slide down.

My idea of a city of individual homes involves a liberal appropriation of land and, therefore, a spreading over large areas and large expenditure of public or private means to realize the idea. A Metropolitan City may embrace a number of semi-detached cities. The engineering part of the great problem of salutary municipal homes is to provide all the conveniences at or near the home, at such cost, so that the working

man of average earning capacity can have the use of them as owner or lessee, and further, to provide ways by which the multitudes can be quickly and cheaply carried from home to working place.

The alternative is set before us to rear good citizens in pleasant salutary homes, with opportunities for mental and moral education, or to breed anarchists, thieves, drunkards, prostitutes and paupers in tenements and slums.

If we will shut out from view the slums of the old part of the cities, Greater London and New York may be cited as partial foreshadowings of what metropolitan cities may be. We see,

1st. A central business heart, crowded even to jostling during the day, but silent and well nigh deserted at night and on holidays. The toilers resting or recreating miles away from the scene of their toil. This heart is the commercial and financial center, which cannot be spread out, but must be condensed in space to a degree that would be intolerable if the hours of work were long.

2nd. In suitable localities, more or less remote from the business heart, we find industrial centers, and of late years these tend to be confined to a narrow belt along railway lines, so that each establishment may have direct shipping facilities.

3d. Residence districts of several grades.

It is desirable that the industrial centers should embrace a variety of works or of departments calling for many classes of labor, in order that all the working members of a family may find employment suited to their age, strength and sex, within a reasonable distance of the home. The working people of such a center can find ample space for separate homes within a half hour's ride by electric car, say three to four miles; employers and short hour workers can afford to ride farther.

These industrial centers and residence districts may be semi-detached in location, but should be so associated in name and government as to subject transportation rates and terminal charges to equitable control.

I have named the two cities—Greater London and New York—as shadows, not examples, in order to forstall the thought that I am suggesting a visionary project, and I claim that practicability is proven by what has already been done. The suggestion is not mine, but it is made by the progress already achieved toward the condition I am endeavoring to set before you. You may see for yourselves the densely crowded business center, where floor space within the favored area is so valued as to have brought into being many storied office buildings, in which buying and selling and financial transactions take place out of

sight of the commodities dealt in. The ware-houses and manufactories are now placed along and near railways, for a switch is a cardinal necessity to a successful business. The small shop on a back street, receiving material and sending out product by dray and wagon is passing away. Building associations and the real estate speculators have joined forces with the street railway to make it possible for the artisan to gain a home in full or prospective ownership, at but a little more expense than renting amid squalid surroundings. These and other forces are working and the work has far progressed already.

Our cities, in their rapid expansions, are working at the problem in an unconscious, unintelligent and therefore imperfect way. What should be done to render progress in city expansion intelligent and adapted to coming conditions?

1. Upon the part of civic publicists, there should be a careful study of the principles of municipal organization and a determination of the form of city government best adapted to American cities. Shall the government be essentially aristocratic? That is, have its executive powers virtually in the hands of one man, the Mayor; giving him power to appoint and remove at pleasure any and all subordinate officers and employes? There are many and strong advocates of this plan of concentrated resopnsibilty. Contrasted with it is that of making the Mayor little more than Chairman of an Executive Committee, the Council. Under this plan, as territory and interests increase, executive functions are subdivided and assigned to Committees of Council. This plan of government by Committee seems to have worked well in the cities of England and Scotland. A third plan is to commit many important fuctions to Boards, more or less independent of Mayor and Council, they often being the creatures of State authority. A fourth plan is a composite of the best features of the three, giving broad powers to Mayor, limiting Councils to legislative acts, giving to heads of Departments a definite term of several years, and grouping them for consultation and initiation of projects.

A chief reason why the first is urged is drawn from the bitter experience of several of the largest American cities, in which the outcome of election of Councilmen by wards has been to fill those bodies with corrupt, or what is equally bad, with incapable men. Combines organized and controlled by unscrupulous bosses have their own way and the objects of Municipal government are perverted. If there be any remedy for these evils in investing the Mayor with autocratic powers and holding him responsible, it must be that it is due to the fact that the Mayor is elected by the voters of the city at large. Precisely the same may be said of a council elected in the same way.

The second system works well abroad, for special reasons which do not exist in America, and cannot so long as we retain our present form of government. Elective positions are so few in number on the other side, that to be chosen to one is esteemed a high honor, though the emoluments be small or nothing; consequently, the best men in the community are ready to take office and serve faithfully. Again, a stricter enforcement of law and a cleaner administration of justice serves to keep corrupt men out of office, or if such get in, to restrain them by salutary fear of consequences. With us, the multitude of elective offices has cheapened them in general estimation to such extent, and corrupt men have had complete sway for so long a time, that candidacy for place in Municipal Legislative bodies, exposes the candidate to suspicion of crooked intent. Especially is this the case if election is by wards.

Some attribute a large share of the evils in city government to interference with elections for city officers by the organizations of National political parties. Of course it is absurd to suppose that a man's views as to protection, free trade or tariff for revenue, have anything to do with his fitness to pass on matters of purely local interest, but there is a decided advantage to be derived from the division of voters upon some line into two nearly equal bodies, for then the independent voters may have hope to hold the balance of power, and it is the independent voter who chooses men for what he at least believes them to be. To the independent voter we must look for restraint—of evil and furtherance of good.

For American cities, I favor: A Mayor, vested with executive powers; a Legislative body in two parts—one elected at large at somewhat infrequent intervals, the other elected by the voters of districts composed of two or three wards, on a combined ticket; Executive department heads to be appointed by the Mayor with approval of Council, but providing ample power of removal for cause, and restriction against removal without cause; in the departments a qualified civil service, elastic enough to cast out incompetence, and rigid enough to hold against changes for personal or political reasons.

How to provide a government for a city of metropolitan dimensions is a matter of difficulty not yet worked out, for metropolis does not need the same amount of governing in all its parts. There are governmental functions which affect all parts alike, such as we usually see exercised by counties; there are others which are locally, but not universally required, such as we see exercised by our city and village corporation.

General policing, courts, criminal and civil, the care of the poor and helpless, asylums for insane, jails and houses of detention, are matters in which the sparsely populated parts of a Metropolis are as much interested per capita or unit of value as the densest. To these may be added main thoroughfares for traffic, and public buildings. Next in possibility of extended interest is drainage, sanitary and surface, but the interest will not be uniform. It will be limited usually by water sheds, and will vary with the use of the ground. Closely following in extent and variation of interest is that of water supply.

More narrowly local, are sewers for complete sanitary or surface drainage, the making, maintaining, cleaning and lighting of streets and many other functions that belong to local affairs.

An organization suited to the conditions presented by a metropolitan city in the United States must of necessity be complex—a goodly measure of autonomy should be reserved to certain component parts, and the more general powers vested in a superior body. The proposition introduced in the Illinois Legislature a few weeks since, to confer special powers of legislation upon a local assembly for Chicago, is by no means absurd. It contains an element of adaptation to obvious needs. The country representative has no appreciation of the needs of cities, and too frequently is influenced by a spirit of hostility to the city. A goodly measure of autonomy is essential to a metropolis.

Building along more familiar lines the functions of the county may be expanded to include all needed by the whole metropolitan district—but the machinery of county government would need to be entirely remodeled whether it be of the representative form, common to the northern and eastern states, or by county court as in the south and some western states. But the essential idea of counties with township organization is the one that seems to be most practicable. But schemes of government are not usually worked out by engineers, and I pass to considerations more immediately pertinent to my audience, and the first is the manner of inaugurating and controlling public works.

In a great city there are several departments of public works, each of which will be of sufficient importance to justify its having a chief, technically and practically familiar with that department and broad enough in his constitution to understand that his department is part of a whole, and to rightly estimate its relation to each other part and to the whole. If so constituted, each chief will be a fit and valuable member of a controlling board of public works.

Such departments may be specified, as :

1. Streets, their laying out, gradation and construction, maintenance, cleaning, sprinkling, and general supervision.

2. Water works, design, construction, extensions, distribution system, operation, and oversight of use.

3. Sewers, studies of system, plans, construction, maintenance, oversight of use.

4. Electrics and gas, all matters pertaining to lighting of streets, public places and buildings. Subways or conduits, construction or use. Also all overhead wires and their supports. Control of measures to ensure public safety as affected by the distribution or use of electricity, gas or other means used for light, heat or power furnished beyond the limit of the premises where it is produced.

5. Buildings, oversight of construction, repairs and care for public buildings. Also supervision of erection, repair and condition of private buildings so far as needed to ensure observance of regulations.

6. Transportation, railways, switches, harbor and docks, and other services by corporations semi-public in character.

7. Parks and recreation grounds; laid out, improved, maintained, and controlled.

8. General superintendent, or chief, to whom may be given the supervision of matters not specially assigned, but better if without departmental duties, who shall exercise control over expenditures, shall sign all contracts, approve all appointments of subordinates, issue all special tax bills aud keep the general records.

If you will consider these departments, you will see that each requires a man specially fitted; also that their inter-relations are so intimate as to suggest a grouping. If these heads constitute a board, then the grouping suggested, points out the appropriate committees to prepare and consider all measures brought before the board.

The heads of departments should not directly have to do with legislation of a general character, but all measures providing for public works, and the manner of their execution; also the estimates of cost should originate in the board, be subject to approval or disapproval by legislative body or councils and mayor, but not to amendment by them without consent of the board. The board should let all contracts, approve of all final estimates, and order the making of special assessments. No measure or order contemplating work to be paid for by special assessment should be recommended or issued without giving parties to be assessed an opportunity to be heard by the board, or a committee thereof, as the importance of the matter may justify. Neither the board or any of its members should have anything to do with the collection or disbursement of moneys.

Some of these heads of departments would by logical necessity be

engineers without its being required by law; others might not, and I would be disposed to require that qualification for Street, Water, Sewer and Electric Departments. Buildings should be given to an Architect. The other positions might be left open on condition that the five named offices are limited to men of technical and practical training; otherwise the last or general Superintendent must be an engineer.

This last should be an elected officer and rank as high as the highest, for he will at times need to be independent of Mayor, Council, or other authority or influence. His function within his sphere is something like that of the Doctor, or Counsellor—legal or spiritual; he will have to give unwelcome advice, oppose pet schemes, and perhaps resist evil. The other heads of departments may properly be appointed by the Mayor, but at a time and under conditions that will farthest separate them from being dictated by purely political considerations.

An organization such as I have sketched will be able to handle the public works of the largest city, and do so economically. Of course there must be a number of sub-departments and many subordinates in each case. What is commonly called engineering work will be done in and under the sub-departments. A prominent feature of this scheme of municipal works organization is specialization.

As a schooling place for young engineers of unformed intentions I do not recommend city employment. It is better by far that such take their first practical initiation in connection with private parties engaged in general practice of city work. There are useful traits or habits of mind that prepare one for department special work, which can be better obtained where strict economy of time, and means are enforced and study made to reach ends by the shortest road, than in any public office.

The unofficial engineering in a great city is greater in volume than the official, and will doubtless increase in volume and importance. For no small part of the extension of improvements must be made under private engineers, representing and serving individuals or corporations, who will develop and improve new quarters more rapidly, more consistently, and more cheaply than is possible under official restraints. And in fact I anticipate that it may be found practicable to turn over to the private practitioner much of the routine and detail work of profiling, cross sectioning, staking out, estimating, and possibly superintendence of construction, especially in case of assessment work. Men who can find, or make and hold, their place in this unofficial practice will be found better qualified to assume the headship of a sub-department than persons exclusively trained inside of the department. And

the same is true for department chiefs; for the outside man has spurs and inducements to special studies and the development of that valuable faculty known as executive ability, that the inside man does not feel. The inside man tends to fall into routine ways, to cling to traditional methods and to resist new suggestions as troublesome innovations.

I have in mind a city which in its early stages of growth was limited to the broken ground adjacent to the river on which it was situated. A practice which became a canon was to make the minimum of street gradient 1½ per cent. In time the city grew beyond the rough ground near the river and extended into the nearly level uplands. It required time and almost drastic measures to check and prevent the extension of the traditional minimum into districts where it involved the heaping up of summits and formation of artificial depressions into a series of short meaningless waves. In fact a considerable area of territory was permanently defaced before a thorough reorganization of departments made it possible to cast aside the arbitrary minimum of the early days.

The coördination and coöperation of official and private engineers, is practicable, for it exists, to a considerable degree, in all large cities. My suggestion is to extend and systematize a relation found to give good results. In my experience a decided advantage results from bringing into contact the private engineer who looks at plans and propositions from the property owners' peculiar point of view, and the official whose point of view may be too high for him to notice little things, or whose vision may be too far reaching for him to notice that he would overtax the resources of to-day in the effort to provide for a distant future. Bringing these elements together is productive of the happy working mean.

The Board of Public Works should be the central authority which forms general plans, and determines the policy to be pursued. Vested with authority to revise, approve or reject all projects for improvements within Metropolitan limits, from the laying of a house drain or the building of a shed to the development of a system of railway terminals or the building of a business palace. This oversight should be so exercised as to afford the maximum of freedom to individual or corporate enterprise, subject always to the condition that the public safety and convenience must be provided for; because the official board is first of all the guardian of public interests.

The plans and policy of a Board of Public Works are subject to limitation of a financial nature. It must cut the garment to suit the quantity of cloth furnished.

Engineers and especially specialists are not qualified to handle questions of finance, but they must familiarize themselves with the limitations which financial considerations set to their plans, and must adapt themselves to such conditions as much as to physical.

Three methods of providing means for improvements under city authority are in use:

1. Borrowing—on credit of Municipality or Sub-division thereof.
2. General and Special funds of Municipality.
3. Assessments upon property benefited.

Many improvements are now being made by private enterprise which bring the engineer close to financial considerations. In my opinion, this practice will greatly increase. To some extent improvements are made in this way, after the streets are opened as public, by consent of the constituted authorities, plans, materials and methods of construction being approved and the work done to the satisfaction of the officials.

Some claim that better results are thus obtained and at less cost than if done in the routine way prescribed for improvements under city authority, as to the probability of such comparative results I say nothing, for no general statement can be made. It ought not to be so, and when it is there is something loose or wrong in the conduct of public work. By far the larger part of improvements made under private supervision is done in anticipation of putting property on the market. To that branch of the subject I will return.

Of the methods of Municipal Works finance named above, an engineer is at times called on to express an opinion. Borrowing is advocated generally upon the plea that the improvements are for the benefit of the future rather than of to-day, therefore, posterity should foot the body of the bills. I have in mind cases in which the only evidence posterity has that improvements were made is the burden of paying for them. Borrowing by municipalities may be advisable or not, just as in the case of individuals or corporations, the outcome may be prosperity or bankruptcy. It is never well to borrow for living expenses or for matters of temporary use. If the use of borrowed money is to create a value or asset greater than its cost, then posterity has no ground to complain, and this is as near the enunciation of a general principle as it is possible to come.

The attempt to provide for improvements out of general and regular municipal revenues will always fail. A large revenue in sight, not specifically pledged, leads to extravagance and wasteful expenditure in every department of city administration, and when a scale of expendi-

ture is once established, it becomes practically impossible to curtail it; as reformers who have undertaken the task well know. The only way to keep fixed expenses down is to keep general revenue within close limits. The ideal plan of revenue might be to fix general revenue at an unalterable percentage of assessed value and shut off all other sources of income. Special funds for special objects, may then be created to which all income, other than property taxes, may be devoted and such special rate of property tax as the people may authorize. The question of such authority being re-submitted to the voters at short intervals. There are many improvements in a city so general in their character that their cost must be borne by the whole city. There are others which with equal certainty are so clearly local that beyond question they should be paid for as local benefits, that is, by special assessments. Though, as I have said, the engineer is not a financial expert or in form a financial officer, he comes very close to the question of special assessments, for his plans must be studied with reference to an equitable distribution of the cost of proposed works. He virtually defines the boundaries of benefit districts, and must see to it that they include all beneficiaries,—with details he has nothing to do. Grading streets and all expenses by way of costs and damages incident thereto; paving all streets, except main thoroughfares and they in part; sewers, almost without exception in great cities; water distribution, except mains and they in part; repairing, cleaning and watering streets, and the removal of garbage, ashes and rubbish, are each and all properly the subject of special assessments; and you will notice that the list includes nearly all the public works of a routine type. A great city is impossible upon any other principle than that the beneficiary must pay for what he enjoys. Equally fundamental is the axiom that—if he enjoys he must expect to pay proportionately. Further, he who lives and prospers in a city must expect to have a scale of necessary expenses approximately in proportion to the size of the city he lives and prospers in.

It is altogether a mistake to put among the supposed reasons why the drift of population is toward large cities, the idea that they can live more cheaply. So far as economical reasons may influence the tide, it is the expectation that more of comfort and pleasure can be had as the result of ones efforts in the large than in the small town or country; and then there is in the minds of some the chance of winning one of the grand prizes of fortune of which they read so much.

The way to realize a municipal millenium is to impress upon the minds of the working people of America that their case is hopeful,

that if they will exercise industry, and cultivate habits of thrift and tastes for home and modest social enjoyments, a good degree of comfort for themselves and a hopeful future for their children may be had, as well under the new conditions of life we are coming to, as has ever been possible.

To dispassionate consideration there is nothing to be said against, but much for, the segregation of people according to their circumstances. The palace and the hovel, if near together, are the occasion of envy and mutual dislike. To such segregation the tendency in the large cities is manifest and irresistible.

You will have already observed that this address has running through it a thread of optimism to the effect that when a tendency toward a certain line of action by large masses of men, and especially if the same line of action is independently adopted in places widely differing in their race and industrial characteristics, then I interpret the facts to mean that a natural development is being worked out, and that such are likely to prove themselves to be of the beneficial order rather than of the contrary. I suggest the study of such unfolding tendencies, so that an intelligent policy be pursued toward them. Now this segregation of classes becomes possible only in large cities, and develops just to the degree that it is possible and therefore may be claimed to be in accord with the nature of men. These segregations superficially appear to follow the freaks or whims of fashion, but, in truth, there is reason why certain quarters should become favored by the rich and why all who can afford it, seek to live near to if not in them. One consideration is the possibility of passing to and fro by public conveyance, without coming in contact with people of filthy or repulsive habits, or passing through burnt districts whose scenes and sounds are not fit for the eyes and ears of wife or children.

Associations of every nature are more and more becoming organized and developed along the lines of congeniality of tastes, identity of interest, accordance in principles. It is, therefore, to be accepted that there will be residential quarters exclusively occupied by the very rich; others by those in well-to-do circumstances, and so by degrees down to the lowest condition that can be allowed to exist.

The city engineering that undertakes to apply an invariable grade and kind of public improvement, to all parts of a town is mistaken and wasteful. True skill adapts itself to conditions and spends in view of probable returns. Beginning with the lay out of the ground: a high grade residence section should have wide streets, deep lots and liberal frontage to lots, with restrictions as to character and position of

buildings. No alleys should be allowed. In other words, for such a purpose the appropriation of land should be lavish to be in keeping with the character of improvements and with the luxurious life of the residents. For well-to-do residence sections the same principles should apply with less width of streets, depth and individual frontage of lots, but the appropriation of ground should be generous.

For the comfortably circumstanced, who must consider limited income, a third scale in appropriation of ground is called for, and for those who must live close, a fourth. A good limit for width of street for fourth class districts is 60 feet, which fixes the least possible distance between house fronts. Eighty feet for lot depth is also a good limit, for the depth of lots should be so small that ordinary sanitary restrictions concerning light and air will prevent tenements on alley fronts. Alleys in rear of lots for the 3rd and 4th grades are a convenience an approach being necessary. The limits named will permit and almost compel a space between buildings at rear equal to that in front. No consideration of adaptation to business purposes need be given in the lay cut of the two first named grades; and the lay out of the last two will be naturally adapted to such business purposes as are necessary and appropriate. Lands along or near lines of transportation by water or rail will naturally come to be occupied for business purposes. Also those immediately adjacent to principal courses of drainage. Along transportation lines the unit of subdivision may profitably be the block, for the tendency is towards a generous use of land area for industrial establishments.

We commonly speak of cities as noted for their commerce, manufactures, art, and seldom think that the most important product of a city is the material for its own continuance and progress,—the boys and girls reared within its borders. No direct measure of the importance of securing the highest quality for this human product is possible, we can best estimate it by considering the reverse side, the horrible consequences of failure. It costs more money to restrain the vicious and care for the wrecks of humanity than to provide safeguards against lapse into vice or against physical degeneracy. We commend the enterprise of the merchant who spends money freely to extend his business; of the manufacturer who throws out old machines and processes, because better may and therefore must be had. We think well of the professional man who holds loosely the traditions of the past and is ready to adopt that which gives promise of better results; why then, should Municipal authorities, and especially the tax-payer, hesitate to meet necessities clearly arising,

by measures suited to the occasion? It costs money and much of it to build a city, also to maintain and operate it; the human as well as physical part. Beginning with the bare site for a city, the first step is destructive of former value; the use of the land to bear crops is at an end. Streets must be opened and given a grade suited to traffic, the adjoining grounds if not in due relation to the street as graded must be brought into such relation by cutting or filling. In this passing from one use to the other, there must be of necessity the loss of former value or damage on the one hand, and the substitution of a new value from adaptation to another use or benefit. When this change is made by the Municipality by enforced proceedings, the adjustment of compensation is one in which the view of the engineer who sees what is to be and that of the owner whose view is backward, are far apart. The Court comes in as an arbiter and is usually strangely oblivious to the possible benefits and ready to take an exaggerated view of the passing values. What shall the engineer do? Break the alignment of the street to avoid buildings, or some tree which has become as the very apple of the owner's eye? Shall he sacrifice the grade and follow the natural surface so that the adjoining property may be at grade without cost or sacrifice? Or, shall he consider that street and grade will remain long after building, tree and owner have passed away?

So much for the official engineer in relation to one part of city extension. How is it with the engineer employed by the owner to lay out his property? Shall he work for one master, his employer; or two, employer and public? or the two and a possible third, himself, as judged by his work in after years? Looked at from this side, it would seem that he has a hard task, but if he will carefully consider he will find that these seemingly conflicting interests will be reconciled if he can hit upon the right thing to do, have the faculty to see clearly for himself and as clearly to set the matter before others, patience to hear and weigh objections and tact to lead those he is in contact with to choose the better thing. Often times he will need to make concession, to do the thing he would not, but let him be careful lest he allow himself to do what he should not. There is often a broad distinction between these.

A street plan must provide first of all for thoroughfares leading from center to center by the most direct available route. Such thoroughfares should be of ample width, 80 to 100 feet, according to circumstances. They naturally come in time to be streets of retail shops, which require a goodly width of sidewalk and the center will be used for street railways, hence a less width than 80 should not be proposed.

More than 100 is unnecessary and adds too much to cost. For business streets a good division is sidewalk one-fifth and between curbs three-fifths. Grades and pavements of thoroughfares must be suited to traffic. For sidewalk paving, artificial stone flagging has demonstrated its fitness and durability,—if honestly laid of good material. It can be put down in most localities for less cost than natural stone flagging. For roadway paving subject to very heavy traffic, trap and granite blocks will be most economical to maintain. Under moderate traffic, that is to say, of weights on wheel, sheet asphalt and vitrified brick are about the only materials now known to be fit. The day of wood street surface has come and gone thrice during my active time, and I think the material has now passed beyond consideration. Sheet asphalt is likely to acquire new popularity and to be laid at a considerable reduction of price. One of the steps to such a consummation being the development of numerous deposits of asphalt in our own country, whence refined asphalt may be distributed to any point and in the form of natural rock to many cities at reasonable cost. A promising field for a limited number of young men is to become expert in the handling of this material under the varying conditions met in its practical use. There is also need for a laboratory investigation of the properties of asphalt so that specifications can be drawn by engineers instead of contractors.

Vitrified brick as a street paving material is rapidly coming to a prominent place. The advance in the modes of manufacture, and towards uniform quality of product has been very rapid, and there is now no reason to hesitate about its adoption for fear of failure. Further progress will doubtless be made in manufacturing processes, and engineers will also come to adopt rational specifications for, and correct modes of using this material. Good material deserves good treatment, and good treatment means good foundation. This remark is applicable to all kinds of pavement.

Rapidly growing cities not having a gravelly soil to begin with are compelled to use for a season the cheapest material that can be had for street surface; the ruling consideration being that it must keep vehicles from miring. These makeshifts hardly come within the province of the engineer, for skill is not required to use them, nor science to discover their weak points. We will charitably pass over these as belonging to a temporary stage. It will assist the passage from use of temporary expedients to more lasting work, if it be understood that streets not used as thoroughfares, and especially side or back streets on which the only vehicles are the delivery wagons, do not require a wide driveway,

which may be reduced to eighteen or twenty feet. For residence streets in general, the narrower the driveway the more attractive the street, if the unpaved surface be planted and well kept.

The practical difficulty in the way of realizing our ideals is commonly financial, but a large part of the glory and honor of our lives depends upon overcoming difficulties, and must come from obtaining desirable results with limited means. The engineer never walks in dreamland, where he might think of what he would do if unrestricted, but in the every day world of what can be done with conditions as he finds them.

Some may think that I have been leading you through a dreamland, because cities grow—we know not how or why—rather than are planned and built. Just so far as this is true, those who have to do with municipal affairs are justly subject to reproach because they are not doing their full duty. The kings of olden time used to found and build cities. In modern times a Baron Hausman had the courage to undertake the transformation of an old city according to a plan. Measures for similar transformations on smaller scale are not uncommon in old world cities. Cannot as much or more be done, under American institutions? Where we mostly have not to tear down old structures, but only to enter open fields and transform them to become the fit abode of American citizens. Many changes would need to come into our governmental ideas before what I have sketched could be realized through the regular municipal agencies, possibly the partial adoption of municipal socialism ; but there is another agency that can be brought into the work. Associated capital, wisely employed, not working from philanthropic motives but for the more sordid one of profit. We have all seen railway companies come boldly into a city and accquire not rights of way merely, but ample terminal and yard facilities ; removing or razing entire blocks of buildings. We are familiar with the remorseless action of a social or moral blight which seems to fall upon sections of cities when, for any reason, they have fallen out of step with progress, which is more destructive of values than a fire and open to no remedy but change of use and reconstruction. In a few cases we have seen a coterie of men join together as a so-called syndicate, purchasing all the property within the limits they have marked out, removing everything in the shape of improvements, wiping out existing streets and by a new lay out transform the ground from inferior to better use, with profit to themselves and benefit to the community at large. Many cases have occurred where large tracts of land have been transformed from agricultural to urban uses by capital used under the skillful direction of the engineer. These enterprises have usually comtemplated eventual

occupancy by the wealthy, well-to-do and comfortably circumstanced classes. A wider and as profitable field is open to those who will provide homes and home places for those who daily toil for daily bread.

All that is required to realize what I have sketched, is a mind to grasp the possibilities of the situation, the ability to set these possibilities before men of means and enterprise, the tenacity of purpose and executive power to carry out a plan. The engineer so gifted will find in every growing city an opening for him, none the less promising if he have one or two competitors.

Cities have characteristics no less marked than men. Some exist and grow because of natural advantages, such as harbor and water transportation, without effort or the assistance of wisely designed and executed public works, *e.g.*, London, New York, Philadelphia, St. Louis. Others have some natural advantages, but are handicapped by lack of these natural facilities for commerce, but which respond to the necessity of providing what nature denied ; *e.g.*, Liverpool, Manchester, Chicago and Buffalo. Others have few natural advantages, but have indomitable courage and enterprise, backed by faith that he who works shall win, they go about the creation of all they want ; *e.g.* Glasgow.

The engineer is peculiarly fortunate who pitches his tent in these latter cities, for the man with ideas is in demand in such places.

I cannot close without suggesting that a very important problem must shortly be solved, that of providing means for the dense masses who crowd the business heart to pass to and fro, within the restricted area. I have stood at a street intersection where street cars at one-half minute intervals during rush hours, were crossing at right angles and, calling to mind the increase of movement within ten years, have wondered what new devices would be required before another ten years. There is a future for engineers in and about cities broader and more inviting than was before the young men of my time, when the chief employment was pioneer work in the wilderness.

Measurements of the Flow of Water in a One-Inch Pipe by Means of the Pitot Tube.

BY W. H. LEDGER, B.E., SYD.,

In the fall of 1894 some experiments * were made by the writer in the Hydraulic Laboratory of the College of Civil Engineering, Cornell University, with a view of obtaining some information as to the availability of the Pitot Tube as a water meter. To this end a small Pitot Tube was inserted in a one-inch brass pipe and the *difference* of pressures on the side of the pipe and at the mouth of the tube was measured during the flow of water by transmitting these pressures to the opposite limbs of a U tube half filled with mercury, the difference of the level of the two columns being a measure of the difference of pressures. This instrument is here spoken of as the piezometer. The scale of this piezometer was graduated to hundredths of a foot. The height of the U tube, and therefore the greatest difference of levels observable in the mercury surfaces, was three feet corresponding to a difference of pressures of 16.38 pounds on the square inch. The water was obtained from one of the main water pipes of the laboratory by means of a 2 inch wrought iron pipe. This pipe was connected to the end of a horizontal cast iron cylinder 1 foot 8 inches internal length and 1 foot internal diameter. By this means a steady flow could be obtained. Stop cocks were inserted in the top of this cylinder for measuring the pressure of the water, and providing means for the escape of any confined air. One of these stop cocks was connected to an open mercurial manometer capable of measuring a difference of 8 feet 6 inches in level of the mercury columns, corresponding to a pressure of 63 pounds per square inch, or a head of water of 111 feet. The greatest head obtainable in the laboratory is about seventy-five feet. For measuring small pressures an open water manometer was used. Both of these instruments were graduated to hundredths of a foot.

***Continuing an investigation begun by Mr. E. J. Fort in 1893.**

The water flowed from the end of the cylinder opposite to the one through which it entered, and at the same level, into the one inch brass pipe in which the observations were made. This pipe was accurately turned to an internal diameter of one inch, the inner surface being made as smooth as possible to prevent the formation of eddies and to insure steadiness in the flow. The pipe was made in sections so arranged that when connected the end of the Pitot tube came in the same transverse plane as the orifices through which the lateral pressure was transmitted. These orifices were four in number and were drilled through the pipe at points 90 degrees apart. They were $\frac{1}{16}$ of an inch in diameter and led into a small brass chamber entirely surrounding the pipe, to which chamber the rubber tubing from the piezometer was attached. The water was allowed to flow into a vessel of six cubic feet capacity placed on the platform of a weighing machine. The time of flow was observed from the second hand of an ordinary watch, this being sufficiently accurate for the purpose. Two regulating cocks were placed in the line of flow, one up-stream in the two inch pipe just above the cast iron cylinder and the other down-stream in the one inch pipe about a foot below the point of observation. By this means the head in the cylinder could be regulated.

Three forms of the Pitot tube were used :—

A. A conical tube, the orifice being in the apex of the cone and the exterior of the tube diverging from the orifice so as to reduce the disturbance of the flow to a minimum. The edges of the orifice were made as thin as possible. The diameter of the orifice was $\frac{1}{32}$ of an inch and of the thick end of the tube $\frac{1}{8}$ of an inch. The length of the tube was $1\frac{1}{4}$ inch.

B. A cylindrical tube $\frac{1}{8}$ of an inch in diameter and $1\frac{1}{4}$ inch long with an orifice the full size of the tube diminishing at an angle of 45 degrees to a diameter of $\frac{1}{32}$ of an inch. (Trumpet-shaped orifice)

C. A cylindrical tube $\frac{1}{8}$ of an inch in diameter and $1\frac{1}{4}$ inches long with a circular orifice $\frac{1}{32}$ of an inch in diameter, the end of the tube presenting a plane surface at right angles to the direction of flow of the water.

Each of these tubes screwed into the side of the end of a small cylindrical tube, also $\frac{1}{8}$ of an inch in diameter, passing through the side of the pipes and connected by $\frac{3}{8}$ inch rubber tubing to the piezometer. In the following pages these tubes are spoken of as *A*, *B*, and *C* respectively.

Experiments with tube A :—With this tube twenty-six observations were made with the tube pointing up-stream and adjusted as

accurately as possible to the center of the pipe. Seven observations were first taken with the down-stream cock wide open. The head of the water in the cylinder was measured by the mercurial manometer. Under these circumstances the highest pressure obtainable in the cylinder was 33.8 pounds per square inch corresponding to a difference of level in the manometer of 3.41 feet, or a head of water of 43.9 feet. Owing to the withdrawal of water from the main pipe in other parts of the building the mercury in the manometer and piezometer was a little unsteady, but not sufficiently so to prevent a mean reading being accurately obtained. Two observations were then taken with a low head of water, the pressure in the cylinder being observed by the water manometer. The up-stream cock was next closed partially and further observations taken, the flow being regulated by the down-stream cock. The up-stream cock was then further closed and the flow again regulated by the down-stream cock. In this way observations were obtained under various conditions of flow, the pressures varying between 4.6 and 68.7 feet and the velocities between 3.65 and 17.57 feet per second.

Experiments with tube B:—Seven experiments were made with this tube pointing up-stream and under various conditions of flow.

Experiment with tube C:—Only four observations were made with this tube, three with the down-stream pressure atmospheric and one with the pressure increased by closing the down-stream cock.

In addition to the above, ten observations were made to determine the actual loss of head due to the obstruction. The piezometer was disconnected from the Pitot tube and attached to the pipe 2½ inches below the obstruction.

The curves in the accompanying figure have been plotted with the velocity of flow of the water in feet per second as abscissae and the differences of level of the mercury in the two branches of the piezometer in hundredths of a foot as ordinates. In the curve marked *A* the results obtained with tube *A* are indicated by a circle, those obtained with tube *B* by a diamond, and those with tube *C* by a triangle.

Measurements were also made with these tubes pointing down-stream, but the variation of pressure for different velocities was so small as to render this method unavailable for metering purposes. With the highest velocity obtainable the difference of pressure in the piezometer corresponded to only 0.025 of a foot of mercury.

What is actually measured in these experiments with the Pitot tube pointing up stream is the "impulse" of the moving water against the

orifice of the tube. The impulse of a jet of water on a fixed flat surface at right angles to jet is* theoretically

$$P' = 2\,F\,\frac{c^2}{2g}\,\gamma' \quad \ldots\ldots\ldots\ldots \quad \text{I.}$$

where

P' = the impulse, or pressure,
F = sectional area of the impinging jet,
c = velocity of the jet,
γ' = the heaviness of the moving fluid,
g being the acceleration of gravitation.

Applying this to the present case we have from the forces acting, and balancing one another at the mouth of the Pitot tube

$$2\,F\frac{c^2}{2g}\,\gamma' = F\,x\,(\gamma''-\gamma') \quad \ldots\ldots\ldots \quad \text{II.}$$

where

F = the area the orifice of the Pitot tube,
γ' = the heaviness of water,
γ'' = " " mercury,
x = the difference of level of the mercury in the two branches of the piezometer.

It will be observed that the area of the orifice occurring in both sides of the equation does not affect the result.

From II we have

$$c = \sqrt{\frac{g\,x\,(\gamma''-\gamma')}{\gamma'}}$$

$$= 20.13\,\sqrt{x} \quad \ldots\ldots\ldots \quad \text{III.}$$

where c is ft. per second and x in ft.

The average value of this coefficient in these experiments was found to be 25.58, the greatest being 26.06 and the least 25.24. The greatest variation from the mean is thus only 1⅓ per cent.

If by h be denoted the height of a column of water measuring the difference of pressure, eq. III may be transformed into

$$h = 1.3\,\frac{c^2}{2g} \quad \ldots\ldots\ldots \quad \text{IV.}$$

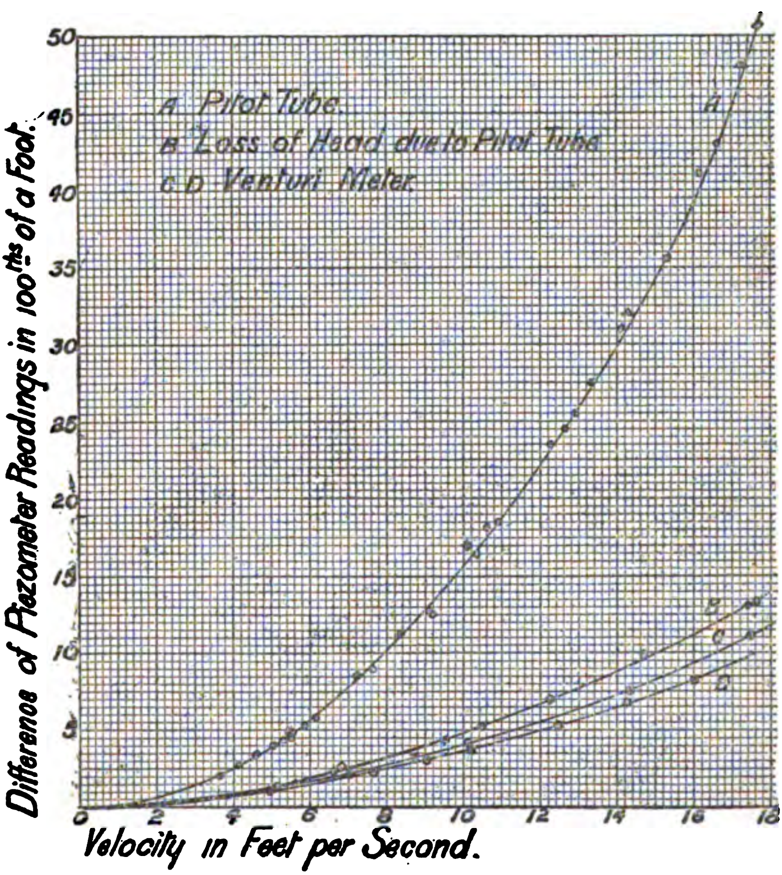
(for any system of units).

It is thus seen that the Pitot tube is well adapted for measuring the quantity of water transmitted under varying conditions of flow. It

*"Mechanics of Engineering," Professor I. P. Church, p. 803.

will work with the same degree of accuracy at high and low velocities and independently of the pressure. It can be made of such dimensions that it will not reduce the flow to any great extent. It is very simple in its construction and would consequently be inexpensive. Its one great draw-back however, is its liability to be choked by material in suspension in the water, a defect which would no doubt prohibit its use to any great extent except for very pure water. Even this defect has a redeeming feature, for, as we have seen, the area of the orifice does not affect the value of x in equation III, and therefore the tube might become partially stopped and still continue to accurately register the amount of water transmitted.

At the same time that the above experiments were made, observations were taken of the loss of head due to the contraction and subsequent



expansion in two small Venturi couplings inserted in the line of flow. These were four inches in length. The internal diameter was one inch at the ends and diminished smoothly and continuously with a reversed curve towards the centre (or "throat") to 0.4 and 0.8 of an inch respectively. The differences of pressures were measured at the upper and lower ends of the coupling at the full diameter of the pipe. Curve *D* represents the results obtained with the larger coupling with the piezometer tubes attached at the points immediately above and below the ends of the coupling. Similar observations taken with the coupling having the 0.4 inch throat showed so great a difference of pressures that a curve could not be plotted on the accompanying diagram. At a velocity of 4.94 feet per second (in the full diameter of the pipe) the difference of height of the mercury column in the piezometer was 0.4 of a foot and at 9.14 feet per second the difference was 1.73 feet.

A few observations were also taken with the large Venturi coupling with one end of the piezometer attached to the pipes 2½ inches below the end of the coupling. The results are represented by curve *C* in the diagram. It will be noticed that the difference of pressure is here slightly greater than that in Curve D. (In the diagram, for "Venturi Meter" read "Venturi coupling.")

TRANSACTIONS

OF THE

ASSOCIATION OF CIVIL ENGINEERS

OF

CORNELL UNIVERSITY

1896

ANDRUS & CHURCH,
PRINTERS,
ITHACA, N. Y.

CONTENTS.

ILLUSTRATIONS.

CONSTITUTION

OF THE

ASSOCIATION OF

Civil Engineers of Cornell University

PREAMBLE.

We, the undersigned, members of the Senior and Junior classes in the College of Civil Engineering of Cornell University, do hereby form ourselves into an Association for the discussion of engineering topics, and the promotion of general information on engineering subjects, and do hereby agree to abide by, and sustain the following Constitution and By-Laws:

ARTICLE I.

NAME.

1. This Association shall be known as the Association of Civil Engineers of Cornell University.

ARTICLE II.

MEMBERSHIP.

1. The Association shall consist of Active and Honorary members.

2. All Alumni of this college and all students recognized as upperclassmen, and registered in the College of Civil Engineering, are eligible to membership in this Association.

3. Any eligible person may become an honorary member by a two-thirds vote of the members present at any regular meeting. Such members shall have privileges of active members except those of voting and holding office, and shall be exempt from all dues.

4. The membership fees of this Association for all active graduate members shall be $1.00 per annum. All money received from membership fees shall be devoted to defraying cost of publication of non-resident lectures delivered before the Association. All other expenses of this Association shall be met by direct tax upon the undergraduate members.

5. A copy of each lecture delivered before this Association shall be forwarded to each member of the Association.

ARTICLE III.

OFFICERS.

1. The officers of the Association shall consist of a President, Vice-President, Recording Secretary, Corresponding Secretary, and Treasurer.

2. The President shall preside at all meetings of the Association and enforce the Constitution and By-Laws, and shall call special meetings at the request of five active members.

3. The Vice-President shall take the chair at the request of the President, and shall act as President in his absence. The Vice-president shall be chairman of the appointment committee.

4. The Recording Secretary shall keep minutes of proceedings of all meetings of the Association and shall post notices for the same.

5. The Corresponding Secretary shall attend to all the necessary correspondence of the Association. He shall be elected from among the Faculty of the college.

6. The Treasurer shall receive all money and dues, and shall pay all bills of the Association, such bills to meet the approval of the Executive Committee before such payments. He shall make a report when called upon by the Association and also when his term of office expires. He shall be chairman of the Executive Committee.

7. The officers shall be chosen by ballot at the last regular meeting of the spring term, from the Junior Class, and shall hold office until their successors are elected.

ARTICLE IV.

COMMITTEES.

1. There shall be two Standing Committees, an Executive Committee and a Committee on Appointments. Each committee shall consist of three members, and be appointed at the beginning of each term by the President.

2. The Executive Committee shall see that the rooms of the Association are ready for occupancy previous to all meetings, and shall transact such business as may be referred to it by the Association.

3. The Committee on Appointments shall make appointments for all literary exercises for each meeting, and such appointments shall be posted at least two days before reading. The committee shall furnish the Secretary with a list of such appointments.

ARTICLE V.

AMENDMENTS.

The Constitution or By-Laws may be amended by a two-thirds vote of all members present at any regular meeting; such amendment to be before the Association at least one week.

BY-LAWS.

ARTICLE I.

REGULAR MEETINGS.

Regular meetings shall be held on Friday of each week, in the Association rooms, commencing on the first Friday after registration week, and ending on the last Friday but one before examination week of each term.

ARTICLE II.

QUORUM.

One-third of the active undergraduate members of the Association shall constitute a quorum. No business can be transacted without a quorum being present.

ARTICLE III.

ORDER OF PROCEEDINGS AT A REGULAR MEETING.

1. Roll Call.
2. Minutes of Preceding Meeting.
3. Literary Exercises.
4. Unfinished business.
   a. Report of Standing Committees.
   b. Report of Special Committees.
   c. Report of Officers.
   d. Miscellaneous Business.
5. New Business.
6. Adjournment.

ARTICLE IV.

EXERCISES.

The exercises shall consist of discussions, memoirs, essays, papers, lectures, and such other exercises as the Association shall from time to time direct.

ARTICLE V.

SUSPENSION OF BY-LAWS.

A By-Law may be suspended for one meeting by a vote of two-thirds of the members present.

H. R. LORDLY,
E. J. FORT,
H. D. ALEXANDER,
Committee.

OFFICERS FOR 1895-96.

President,

HARRY K. RUNNETTE.

Vice-President,

JOHN H. LANCE.

Treasurer,

DANIEL Y. DIMON.

Corresponding Secretary,

PROF. CHAS. L. CRANDALL.

Recording Secretary,

GEO. W. ENOS.

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Members of the Association of Civil Engineers
of Cornell University:

In the college year '92–'93 the Association of Civil Engineers of Cornell University was given a new start in life.

Many of our members are unaware, and others have forgotten, that at that time the constitution was revised so as to make all the graduates of the College of Civil Engineering equally eligible to membership with the Seniors and Juniors, and indeed the graduates were urged to take an interest in the newly formed, or rather revived Association, that the bond between acquaintances in college might not only be kept up but strengthened as time passed and experience was gained. It was no doubt thought (and rightly too), that interchange of views and experience, through the medium of papers by our graduates to appear in the "Transactions," would be a most efficient means of making this bond lasting besides being of mutual benefit in the practice of author and reader alike.

The undergraduate member was to profit through the reading of such papers, at the meetings of the Association, before their publication in the "Transactions."

In so far as we know our graduates are too busy with the daily trials and emergencies of practical life to give us, except at rare intervals, the papers that were evidently looked forward to in the reorganization of the Association in '92–'93.

It would seem that out of the large number of our graduates, occupying, as our recent census shows, important offices in all parts of the country, we could gain a few papers each year to be read by some member and discussed by others at one of the weekly meetings, when non-resident lecturers were not occupying our attention.

It may be suggested that the members who could and would write such papers need a personal solicitation to get from them what is desired, or it may be that they are not aware that the undergraduates here at Cornell would appreciate highly any favors of this nature. It is hoped that any alumnus to whom the "Transactions" go, and who chances to read this will take it for the personal solicitation of the undergraduate members, and at the same time a statement of their needs.

During the year Professor Fuertes has compiled a census of our graduates and any alumnus looking over the record—without a parallel in any like institution—may well be proud that he is such. The record includes eight railroad presidents, eight corporation presidents, seventeen city engineers, five consulting engineers, nine contractors, thirty-seven chief engineers, managers, superintendents of bridge companies, railroads, water works, iron and steel works and other manufacturing establishments; twenty-two professors, associate professors and mechanical and mining engineers, bankers and merchants, in almost every case very successful.

It is to these men that the "Transactions" are annually sent and from whom brief papers on subjects of their choosing would be so highly valued.

It may be appropriate in this connection to call attention to the Fuertes Prize medals, one of which is annually awarded to that graduate of "the College of Civil Engineering" who shall write the best paper on an engineering subject, and the other to that undergraduate who shall obtain the highest standing during his senior year: For the year 1894–95, the graduate medal was awarded to Mr. John F. Hayford, for his paper describing methods of work on the Mexican Boundary. The undergraduate medal was awarded to Mr. A. L. Colsten.

To the present Juniors, in whose keeping lies the welfare of the Association for '96–'97 a few words may not be amiss.

Of the essentials to a successful year you will concede an early start and a strong start to be of the first importance. If you can have papers every week of the Fall Term with discussion on each paper by appointed members, much will be accomplished. The discussion in most cases, will be as valuable as the paper itself as it always brings out the salient points of any article.

Articles in the current engineering magazines and in the Trans. Am. Soc. Civ. Eng., studied and read at meetings by an appointed member, would be of much benefit—also some of the papers translated in the former course in technical reading have been suggested by some of our professors.

As to the matter in the present "Transactions," its genuine merit speaks for itself,—the wealth of engineering literature in the following pages keeps up the high standard of our publication and the articles are by men in the first walks of professional life. The fact that we have such eminent non-resident lecturers attests the fact that the college is active and progressive with the times.

For getting out the "Transactions" in such excellent form the

hearty thanks of the Association are extended to Professor Crandall and to Mr. William Mackintosh together with the other members of the Publication Committee for their indefatigable labors attended with the numerous worries and vexatious delays which only those who have done similar work can appreciate.

The chair on behalf of the Association thanks the other retiring officers for their sacrifice of time and pleasure in working for our interests.

With the best of good wishes for the ensuing year the present Seniors will as graduate members henceforward hope for the continued success of the Association.

Harry Kerr Runnette.

May, 1896.

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ELECTED AT THE LAST REGULAR MEETING, MAY 15, 1896.

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The address on the line with the name is the home or permanent one; that on the line below is the business or temporary one. When only one is given its position is an index of its permanency, except when forced from one position to the other by lack of room. The addresses have been furnished by the members, each for himself, and nearly all since April 1. In a few cases the data has not been furnished by the member himself, but unless recent and apparently reliable it is placed in italics. All changes in address should be promptly reported to the Corresponding Secretary.

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In Memoriam.

C. P. AYLEN, C. E., '76,

Was born at Aylmer, Canada, on October 17, 1854, and after making a brilliant record as a young man of great ability, was graduated in 1876, one of the most promising members of his class.

It has not been possible to trace his career in great detail; but after considerable correspondence the following facts have been ascertained: In 1877 he was engaged in the location of the Pontiac and Pacific Junction Railway, west of Ottawa, Canada. From 1878 to 1883, he was in the service of the Dominion Government, making surveys in connection with the location of the Canadian Pacific Railway and in making a survey of the French river northwest of the Georgian Bay, Lake Huron. From 1884 to 1888 he was engaged in the construction of the Canadian Pacific Railway in British Columbia. From 1889 to a short time previous to his death, he was employed in making surveys for the Dominion Government, in dividing up the northwest territory of Canada.

Meager as this information is as to detail, it will be seen that Mr. Aylen was busy with important trusts throughout his professional life. It is, indeed, lamentable that a young man of so much promise should be cut off so early from a field of usefulness, in which his clear brain, executive ability and manly, honest character gave promises that his untimely death have ended. He will be remembered by his classmates as a man of rare social qualities, witty, straight-forward and as a brilliant, successful student, and companionable and fair-minded man.

THE TRANSPORTATION OF SOLID MATTER BY RIVERS.

WILLIAM STARLING, M.A.S.C.E.

Chief Engineer Mississippi Levee District.

In planning works of river-improvement, it is indispensably necessary to have an intimate acquaintance with your river. An intelligent physician, before undertaking the conduct of an important case, must know his patient's temperament and constitution, his idiosyncrasies and tendencies, his previous life. Not only will minute inquiries be instituted as to these points, but as much as is possible will be ascertained about the patient's parentage and ancestry.

Now rivers are no more alike than human beings. As it may be said of men that no two of them have exactly the same constitution and the same peculiarities, so no two rivers are precisely alike in regimen. As remedies which would be beneficial to a plethoric patient might be fatal to a weak or anaemic one, so devices which would tend to the betterment of clear streams, for instance, might be highly detrimental to muddy ones; and plans of improvement which would be very rational when applied to rivers of flat slope and feeble current might be absurd if attempted with impetuous streams, with a superfluity of destructive energy. A river debouching from a mountainous region, the slopes of which are bare and soft loam, into an almost limitless plain, like the Yellow River of China, is governed by very different conditions from those which prevail in the case of the Mississippi, with a long and comparatively quiet course, with numerous and large clear tributaries, and with a flood-plain which is confined within narrow limits by hills. If a system of cut-offs be considered beneficial to the sluggish Theiss, with its tortuous course and its slope of one-tenth of a foot to the mile, it does not follow at all that it would be applicable, or even practicable, in the case of a stream which has ten times its volume, three times its slope, and a current which is already too great for the stability of its banks. On the other hand, the system of levees, which is thought to be correct practice on the Mississippi, might not succeed with the

Yellow River. A plan for reducing the high-water plane by outlets, which might be rational enough as applied to a clear stream, would probably result in actual deterioration of the channel with a muddy one.

A thorough familiarity with each river is only to be acquired by intimate personal acquaintance, as it were. There are many leading principles, however, which have guided the development of all streams; and there are others which have shaped the career of particular classes of rivers.

There are several ways in which rivers may be classified. For the present purpose, it will be convenient to consider them as divided into sedimentary and non-sedimentary streams. A sedimentary or silt-bearing river is one that flows between banks of friable earth and in a bed which is, wholly or in part, of its own formation. It has almost always velocity enough, in some parts of its course, to erode the soft material through which it runs, and to hold it in suspension. It is therefore apt to be more or less muddy at all stages. Its channel is shifting and unstable. Non-sedimentary or clear streams generally flow over rocky or gravelly bottoms, have stable banks, and are muddy only during freshets. A river may be clear in one part of its course and silt-bearing in another, as when a mountain stream runs through an old lake-bed. Roughly speaking, the two divisions correspond fairly well to the designations of clear and muddy rivers.

The characteristic behavior of sedimentary rivers depends on their ability to take up and transport the solid matters of which their bed and banks are composed, up to a certain point; their inability to exceed that limit; and the compulsion which lies upon them to drop the burden which they have assumed, or a portion of it, when their transporting power is reduced. Non-sedimentary streams do indeed, in times of freshet or after a heavy rainfall, carry loads, even great loads, of solid matter, but the latter are not derived from the bed. They come from the neighboring hillsides, or are brought by the tributaries, the creeks or the rills which furrow the adjacent plain. The storm over, the contributions of earthy detritus cease and the river becomes clear. It is otherwise with the silt-bearing streams. With these, the lining of the channel is so light and friable that a small velocity is sufficient to enable the current to pick up a certain portion of it, and it is said that even a less velocity will suffice to carry the load when once lifted.

The suspension of solid matters in fluids is a subject which is very little understood, and which, indeed, has been very little discussed, when the great importance of it is considered. It might be said without exaggeration that the whole plan of the improvement of a sedi-

mentary stream depends upon the laws governing the suspension and transportation of the burden of solids with which it is loaded; and particularly upon the relation which exists between transportation and velocity.

It is conceded on all hands that there is such a relation; that the greater the velocity the greater the load of sediment that a stream can carry. It is not conceded that the stream *does* always carry the whole burden of which it is capable; and the exact nature of the relation between the two quantities has never been clearly developed. There is indeed a surprising number of obscurities and ambiguities surrounding the subject which there has been very little effort to clear up. You will find it broadly stated in a reputable text-book that the transporting power of a current varies as the sixth power of the velocity. You will also find it affirmed by eminent engineers that the transporting power varies as the square of the velocity. Both assertions are true. If two objects be of different size, but of the same material and of similar shape, as two cubes, double the velocity will move 64 times the weight. If the objects be of the same size and form but of different specific gravities, then double the velocity will move only 4 times the weight.

First of all, it cannot be affirmed without reservation that even still water, or what is apparently such, will not hold in suspension solid matter for a limited or perhaps even for an indefinite time. Perhaps there is no such thing, in our ordinary experience, as absolutely still water—that is, water free from internal currents. Again, there is no such thing in existence as a perfect liquid. All natural liquids have a certain degree of viscosity. Water is a gross and thick fluid compared with some others. Matters which remain indefinitely suspended in water, drop immediately to the bottom in them. Thirdly, there are great differences in the attractive forces between different liquids and different solids. Fourthly, there is a very large range of variation in the purity of the different specimens of natural water which we meet in our daily practice; and suspending power is considerably influenced by the presence of mineral matters in solution. Every day we see familiar examples of finely-divided solids suspended, for an indefinite time, in liquids to which they are not known to bear any chemical relation—as in the various colored inks of commerce. The power of suspension, in this case, seems to depend mainly upon three points—the specific gravity of the solid, the state of division of the same, and the degree of viscosity of the liquid. According to Mr. Carl Barus,* solid particles

*Bulletin No. 36, of the United States Geological Survey.

may readily be obtained so fine that their rate of subsidence is practically infinite—and this is distilled water. In a given case, turbidity continued after five or six years. Even in the case of ordinary river sediment, General Comstock relates that he has put clay in a glass tube with water and hung it up, and that it took three weeks to subside.

It is well known that the addition of a small quantity of gum to water increases its suspending power, and enables it to hold a certain quantity of coloring matter, as ink-powders, suspended for an indefinite time. On the other hand, even a small quantity of acid, salt, alkali or foreign material in general, dissolved in water, will diminish the suspending power and increase the rapidity of subsidence to a marked degree, sometimes even many hundred fold.

The problem which mainly interests the engineer, however, is the suspending power not of still, but running water. This may be considered from two points of view, namely, as the suspended matter is homogeneous in composition and is composed of particles uniform in size, or as it differs in one or both of these respects.

So far as specific gravity is concerned, there is no great disparity in the various substances carried by ordinary rivers. These are usually gravel, sand and clay. The two former are generally of similar composition and differ only in size. The latter is somewhat lighter. The difference, however, is not so great that it cuts much of a figure in comparison with the infinitely more important quality of state of division, or fineness of comminution.

But, first of all, why should moving water bear up solids at all, or at least solids of such size as, it is conceded, would speedily drop to the bottom if there were no current?

Given a perfectly smooth channel and a uniform velocity, why should not a heavy body, dropped into a river at the surface of the water, go to the bottom, in such an interval of time as the laws of gravity prescribe, taking into account the resistance of the water due to the velocity of descent of the solid body and the retardation caused by the viscosity of the water? The time of descent would not be affected by the velocity of the current. It is an axiom of dynamics, that a force acting in a certain direction still continues to act in that direction, despite the simultaneous action of a deflecting force. The only difference would be that the line of descent would not be vertical, but inclined, and that the body would strike the bottom at a point down stream from where it had been dropped into the current.

The assumption of a uniform velocity, however, is not at all in accordance with observed fact. There are innumerable variations of

velocity in every stream, great or small. Ordinarily, water is conceived to move in a series of parallel fillets, each of which has its own velocity. This is the most unfavorable supposition for the suspension of sediment that can be adopted. Even on the assumption, in order to account for the phenomena of suspension, M. Dupuit* has elaborated an ingenious theory which has been adopted by many engineers of distinction.

M. Dupuit begins by calling attention to the fact that the cause of the phenomenon is not the velocity of the water, though this circumstance nearly always accompanied it. The state of rest, says he, is, indeed, but a mathematical abstraction, which can be conceived, but cannot be realized, because all bodies participate in the double motion of the earth. The stagnant water of ponds, which is stagnant only to the observer placed on the earth, may have an absolute velocity more considerable than that of the most rapid torrents. The power of suspension depends then, not upon the absolute velocity of fluids, but upon the relative velocity of their molecules. If all the fillets had an equal velocity, they would behave to a solid body as if they were at rest. M. Dupuit then refers to an experiment, the rotation of a mixture of solids in water, in which the surface fillets which had the greatest absolute velocity, were found not to be those which held the most solid matter in suspension. It was the lowest layers, which, with a less absolute, have yet a greater relative velocity. It is the same thing with large bodies of water.

M. Dupuit then considers the case of a body floating on the surface of a stream, the fillets of which move with a velocity which increases from the banks to the centre, the body being placed close to the shore. This body, falling, as it were, on an inclined plane, has a tendency to acceleration, which is always present, and which is only kept down by the resistance of the water. Now the velocity of the solid must always be greater than the mean velocity of the adjacent fillets, otherwise there would be no resistance. Therefore, the solid will, on the whole, gain on the current. The greatest part of the liquid in front of the body is thus compelled to pass behind it, a very small part of that which is behind passing to the front; so that if we consider the relative velocities of the fillets which envelop the body with respect to the velocity of the body itself, it will be seen that on the side next the bank a retrograde current will be set up, much more considerable than the direct current on the side toward the center of the stream.

*Études sur le Movement des Eaux, p. 217, (ed. 1863).

Hence arises a lateral pressure from the body which pushes it toward the more rapid fillets, in which it would find less resistance.

"This conclusion," says M. Dupuit, "is confirmed by every-day experience. We see floating bodies detach themselves from the banks and place themselves in the axis of the current. Now if we imagine a solid body plunged into a current, we shall find that the relative velocity of the fillets in the vertical direction will, in the same manner, engender a pressure which will carry it from below upward. This pressure may be less than the weight of the body, which will then descend until it meets a stratum where the pressure and the weight are in equilibrium, which will necessarily happen if the current is deep enough; for in proportion as it descends, the relative velocity of the fillets increases, and with it the pressure from below upward. If the pressure, on the contrary, be greater than the weight of the body, it will raise the latter to the strata which, although possessing a greater absolute, have relative velocities which become more and more feeble, so that the solid body will necessarily stop at a certain height.

This beautiful and ingenious demonstration does not seem at all adequate to explain the observed phenomena. Let us take the case of a cube of stone, of an edge of one inch, and of twice the specific gravity of water. On submerging this, it will lose half its weight, as the expression is, so that it will require a force equal to the weight of one cubic inch of water, or 0.036 pound, to uphold it. The pressure of running water against a surface is equal to the weight of a column of water whose base is the area of the pressed surface and whose height is the height due to the velocity. If a be the edge of any cube of the given specific gravity, this pressure will be expressed by the term $0.036\ a^2h$. The weight required to hold up the submerged cube will be $0.036\ a^3$. Equating these expressions, we shall find $h = a$. In the present case, h will be 1 inch, or 0.083 foot. The velocity due to this height is about 2.31 feet. This therefore would be the effective velocity required to hold in suspension a cube of 1 inch, if it were applied directly upward. In the supposed case, it would require a *difference* of velocity of this amount on the two sides of the cube (the upper and lower side) to produce this effect. On the supposition of a variation of velocity in vertical planes, rarely or never realized, the difference in velocity at the upper and lower sides of the cube would be only about 1–60 of a foot, or about 1–140 of that required. To sustain a cube of the size of the smaller particles of sand and clay encountered in the sediment of the Mississippi River, say of an edge of 1–3000 of an inch, a force would be required represented by a column of water whose

height is 0.000028 foot. The velocity due to this height is about 0.04 foot or 1–25. Supposing the velocity in vertical planes to vary as before, to the extreme observed limit, we should have for the difference between the upper and lower sides of a cube of 1–3000 of an inch, an effective velocity of 1–180,000 of a foot, or 1–7200 of that required. The smaller the particle, the greater would be the inadequacy of the means proposed to sustain it.

We cannot then accept this theory as a sufficient explanation of the facts. We must be struck, however, by the justice of Dupuit's remarks as to the small effect to be ascribed to the agency of mere velocity. If we could conceive such a thing as a prism of water rushing down a smooth channel with a velocity which should be absolutely uniform in every part of the cross-section, it is evident that a heavy solid body dropped into such a stream "would never know the difference" whether it was in a still or a moving liquid until it reached the bottom, when it would be rolled by the action of the water, if the velocity were sufficient. But it would not remain suspended.

An explanation which is simple, sufficient and satisfactory, and which has been generally adopted by the engineers of the Mississippi service, attributes the suspension of solids to vertical currents, engendered principally by the irregularities of the bed. It is well known that the bottoms of sedimentary streams, even in their smoothest parts, are composed of sandwaves, giving an undulating profile, and of course causing a disturbance of the movement of the water even to the very surface of the river. For it is known that the influence of the slope of the water-surface affects every particle of water in the cross-section, and causes it to move down-stream. If a particle moving along the bottom encounters a reverse slope, it will be deflected upward, and thus cause a vertical current. Says Professor Chamberlin: "Stir up a little trash in the bottom of any deep pool in a brook, and see how readily it is borne up the slope of the pool-bed, and out over the shallows below. Disturb the bottom-mud above a dam, and watch it ascend the steep slope and pass over the weir.

"The bottom layer flows up and down according to the inequalities of the bed, while the top layer declines more uniformly with the surface-slope. In proportion as the stream is rapid and crooked, these layers exchange places, and there is a tortuous upward and downward flow, in addition to that directly enforced by the bottom inequalities, though all largely due to these." *

* Report of the United States Geological Survey for 1883–4, page 132.

The irregularities caused by ordinary sand-waves, however, are probably small compared with those which proceed from the violent processes of erosion and projection of material which are of hourly and momentary occurrence. The extreme energy of the vertical currents caused by these disturbances of the bottom are well shown to the eye, in the case of the Mississippi, by the "boils," eddies and whirlpools which characterize the surface of that powerful river. According to excellent authority, at all speeds except the lowest, rotating eddies are always formed by the roughness of the solid surface, or by abrupt changes of velocity distributed throughout the fluid. It has been found by Professor James Thomson that there is a circuit of transverse currents in the cross-section of a river in a bend, flowing outward on the surface and inward (that is, toward the center of curvature of the bend) on the bottom—a sort of undertow, or vertical eddy, which produces important disturbances of the flow. These conclusions are not the result of theory alone, but were confirmed by experiments made with light immersed particles, such as specks of aniline dyes. To vertical currents also is attributed the well known fact of a diminution of velocity at the surface of every stream.

The bottom of the Mississippi is known to be very irregular. In the bends, where caving is constantly going on, and immense loose masses of earth are continually projected into the stream, the disturbance of the current is extremely violent. On the bars, where, at flood-time, deposit is taking place, the process is more quiet and less fitful, but still a constant change is progressing. Soundings taken by the steamboats as they cross the shoals, show irregularities at every cast of the lead. To the experienced eye, these alternations of bottom are all revealed by indications at the surface. The "break" of a bar or sand-roof or of a sunken wreck, is perceptible to the eye of the pilot at a depth of twenty or thirty feet or even more. The whole current of the Mississippi is nothing but a series of "boils" as they are significantly called, or vortices, which resemble nothing so much as the ebullition of water; whence their name. A simple inspection of these is enough to convince one of the violence of the disturbances which give rise to them. An exacerbation of the condition is immediately indicated by an increase in the violence of the ebullition, which then becomes extremely powerful and turbulent, and conveys the impression of the exertion of mighty forces. At such times they bring to the surface vast quantities of water densely charged with sediment, to an extent exceeding probably by many fold the average amount held in suspension by the stream. It is found by those who have had the opportunity of observing the phe-

nomena of violent erosion, as seen at the ends of crevasses, etc., that the turbulence of the "boils" is a faithful index of the work of destruction going on beneath. The forces brought into play in these ways are incomparably more energetic than those which may be supposed due to any difference in the velocity of adjacent fillets. They furnish a sufficient explanation of the suspension of solids in moving water, and it seems hardly worth while to look further.

The connection of ordinary longitudinal velocity with suspending force is not so evident. It is generally admitted that there is some connection between them, but it is not by any means agreed what the connection is. In the discussion of the question, it has usually been tacitly or expressly assumed that the sediment was homogeneous both in density and state of division—and it has been attempted, on this hypothesis, to show a relation between the quantities carried at one velocity and at another. Conclusions deduced from such reasoning can be valuable only as the given circumstances correspond to the hypothesis. This, in actual practice, can never be the case. Sediment is never homogeneous. In the Mississippi, it is composed of fragments of all kinds, from sandy particles of a diameter of 1–50 or 1–100 of an inch to minute clayey particles of less than 1–3000 of an inch. At one time the Ohio is the prevailing river. At another, the Missouri may contribute the greater part of the water and nearly all the sediment; and the solids carried by the two streams are widely different in character.

Let us suppose, however, that the sediment is of uniform quality and size. What will be the effect of changes of velocity on the suspending power? First, let us take the vertical velocity. If a filament of water 1–3000 of an inch in diameter, with a velocity of 0.04 foot per second, will bear up a cube of stone of the same diameter, then a prism of four such cubes superposed, one on the other, making four times the weight without increasing the area of the base, could be carried at a velocity of 0.08. In other words, the case proposed conforms to the general principle of work and energy, and the work performed is in proportion to the square of the velocity. In the case of the Mississippi at flood, with a mean depth of 60 feet, supposing the sediment to bear its usual proportion to the water of about 1–3000 by bulk, the depth of each prism to be borne by the several filaments would be about 1–4 of an inch. To bear this weight, a velocity of 1.1 foot per second would be required. This of course supposes the whole velocity to act directly upward, and the work to be expended upon the face of the prism, without loss.

Were the exact amount of velocity expended that is required, the

velocity would be exhausted by the resistance. But only a small part of it acts in a vertical direction, the remainder acting parallel to the bed of the stream. The relations between the total velocity and its vertical component can not be precisely determined. Nevertheless, there are relations between the two, which, like other relations in great rivers, subject to violent and irregular mutations of regimen, are not capable of being reduced to a rule which shall be rigorously applicable to every case, yet may be true in the grand mean. It may fairly be supposed, for instance, that at a given place and under given circumstances which are not materially changed during the time of observation, the vertical component is proportional to the total velocity. If this be true, then the amount of sediment which *can be* carried at a certain place and under unchanged circumstances will be proportional to the square of the velocity. This statement, however, will not or need not be at all applicable to any other place, or to the same place under altered conditions.

It does not follow that because a stream is capable of carrying a certain quantity of sediment it will therefore be always charged to its full capacity; or that if it be loaded with all that it can permanently transport, it may not temporarily take up, and carry, for a limited distance, a load in excess of that capacity, nay, several times as great. This matter is worthy of discussion, because very great issues have turned upon it; no less, in fact, than the whole scheme of improvement of the Mississippi River. The late Mr. Eads bore a prominent part in this controversy. His position was, as stated in his own words, "that the power to transport sedimentary matter depends upon the speed of the current, and increases with the square of the velocity." A corollary to this proposition was that the river is at all times charged with the quantity of sediment due to the velocity; accordingly that if the velocity remain constant, the river can not by any means acquire more; if the velocity increase, the river will proceed to scour the bottom and thus load itself to the limit; if the velocity decrease, the water will immediately drop a portion of its burden, from sheer inability to carry it further. Mr. Eads denounced as fallacious "the popular theory advanced in many standard works on hydraulics, to-wit: that the erosion of the banks and bottom of streams like the Mississippi is due to friction or impingement of the current against them." "A certain velocity," said he, "gives to the stream the ability of holding in suspension a proportionate quantity of solid matter, and when it is thus charged it can sustain no more, and hence will carry off no more, and therefore cannot then wear away its bottom or banks, no matter how directly the current may impinge against them."

The inference which Mr. Eads drew from these premises was that the plan adopted by the Mississippi River Commission for the protection of caving banks by direct revetment was radically wrong. The reason that banks were eroded, according to his view, was that the dropping of a part of the load of sediment on the wide shoal above and the subsequent acceleration of the velocity during the steep fall from the shoal to the caving reach below had made the stream hungry, as it were for sediment. Hence the erosion. His remedy for the evil was to build contraction-works at the wide place above, which should prevent the velocity from decreasing, and thus should keep the river loaded to its full limit. It could then acquire no more.

A part of this argument is undoubtedly correct. In the opinion of most or all river-engineers, however, Mr. Eads carried it too far; and especially he erred in ascribing no influence, however, to angle of presentation of the bank.

The idea that a stream may be, so to speak, saturated with sediment, is not a new one. When understood within proper limitations, it may be accepted as true. These limitations are, first, as to quality of sediment, especially as to fineness of division; second, as to constancy of direction of the current; and thirdly as to uniformity of bottom. With these qualifications, the principle of "saturation" may be stated as follows: Given, sediment of uniform specific gravity and fineness; given also a reach of river which is straight, or so nearly so as not to oppose any sensible obstacle by its change of direction; given also a bottom of uniform character, rough or smooth; if a current flow through such a reach, loaded to its utmost capacity, in proportion to its velocity, supposed to remain constant for the time considered, it will not, of itself, acquire any more. The statement in this form is a mere truism; yet it cannot be modified without danger of misapprehension or error. If the character of sediment be changed in any respect, it is obvious that the rule will not hold good; if the angle of impact be made so abrupt as to make the line of application of the force of the current strike the bank at an angle favorable to abrasion; or if the bottom be so changed in its nature as to convert a greater or less proportion of the total velocity into vertical currents; the rule will obviously be inapplicable.

If a quantity of loose earth be precipated into a stream already loaded to a state of so-called saturation, what will be the result? Will it sink immediately to the bottom? Experience says, No. Even though the velocity be small and the sediment heavy and coarse sand, it will yet be carried a short distance. The definition of saturation, as the term is

applied in this case, is, that condition in which the upward pressure on the surfaces of the particles of sediment is in equilibrium with the weight of the particles. Without a change in the conditions, the stream cannot therefore permanently support any more. The viscosity of the water, however, opposes a certain resistance, which may be, as we have seen, very great, and which is always appreciable. The descent of the particles will therefore be retarded, while in the meantime the water of the stream is making progress toward its mouth. The sediment will therefore be carried a certain distance downstream, and then, without any change in the conditions, will be deposited.

The ordinary phenomena of "caving" banks, are confirmatory of this view. The current, encountering the bank at an abrupt angle, at the foot of a bend, dislodges the friable earth of which it is composed, by a purely mechanical action, like the blow of a pickaxe or of a hydraulic jet. This it will do though it be already loaded to the point of permanent saturation, provided its fluidity be not materially impaired. The earthy matter thus committed to the water will not sink at once to the bottom. It will be carried a certain distance, long or short, and be then dropped, even though there be no slackening of the velocity. By hypothesis, the water was already loaded with all it could permanently transport when it acquired this additional burden. The latter, therefore, must be only temporary. Still more, must it be laid down when the velocity is diminished. Now the velocity in a bend is not immediately checked by the impact of the water against the bank. The acceleration due to the fall from the shoal above contends with the resistance of the bank. Eventually, a point will be reached when the destructive energy of the current is exhausted, the velocity reduced and the angle of presentation of the bank less. This locality is the reversion-point where the bend joins the next bend below. Here several circumstances conspire to lessen the velocity—the shock lately experienced, the great size of the cross-section and the oblique course of the current as compared with the axis of the river. Here then the last of the additional burden will be thrown down.

But again, even directly opposite to the stretch where erosion is going on with violence, there is comparatively quiet water, on the shallows of the opposite bar, where deposition is continually progressing. This is especially the case under the lee of the point, where, accordingly, considerable accretions are made to the shore already existing ; so that as the bank-line recedes on the one side, it advances on the other.

It has already been remarked that there is a lateral circulation of currents in a bend which is centrifugal at surface and centripetal at

bottom. The latter current carries with it a part, at least, of the detritus accumulated at the outside of the bend, and deposits it on the inner side, by reason of the reduced velocity which prevails at the latter place. The deposition of sediment is therefore continuous from the head of the caving reach to the shoal at the reversion-point below, where, in a region of deep and successive bends, it still proceeds, but on the other side of the river, and forms a series of bars joined to the great shoal at the "crossing."

The doctrine that a sedimentary river is always loaded to its full capacity is a very plausible one, and when understood with proper reservations it is probably nearly true. If a stream be overgorged, as after being loaded with the detritus from a caving bank, it will soon drop its surplus burden, but will still remain charged to its normal capacity. If it be under-charged, experience teaches us that it will, sooner or later, take up more sediment, and, in the case of an abrupt change of direction, will even acquire a superfluous load. Whether it will, in a straight reach, and with a smooth bottom, take up all that it is capable of transporting, is not so certain; for excellent authorities assure us that it is easier to carry a burden than to pick it up, and the ordinary sand-waves do not afford salient points of attack. As the time for action is short, namely the time required to pass from one shoal to another, it is possible that the river is at times undercharged, at other times it is overcharged. The disposition of water to acquire sediment, if the velocity be sufficient to carry it, is one of the commonest subjects of observation. In the Ganges canal, at Roorkee, Mr. Login observed that, "however clear the water entered at the head of the canal, it rapidly took up large quantities of silt from the channel. In the cold weather, the observer could see clearly a rupee in ten feet depth at the head of the canal; at the sixth mile it could not be seen at five feet, at the twentieth mile at three feet, and at the fortieth mile the water was full of silt." It is to be observed that the velocity of the water in the canal did not increase during this transit, but actually diminished, at a continuing rate. It was 5.10 feet per second at the beginning, 4.48 at the nineteenth mile and 3.46 at the fiftieth. It is evident then that a considerable time is required for a full load to be taken up.

It has been announced by the Mississippi River Commission as a principle, that if, in a sedimentary stream, the current be checked at all, a deposit will ensue. This assertion may be admitted without conceding the position that the river is always fully loaded. To take the calculation shortly before made, to support the ordinary quantity of sediment carried by the Mississippi, a vertical current of a velocity of about

1.13 feet per second is required. With this velocity, each filament of a diameter of 1–3000 of an inch would support 750 particles of the same diameter, or a filament 1–4 of an inch in diameter would support a cube of the same size as itself. This is the maximum cube that can be carried by this current, but a river a mile wide can bear up 250,000 of them before being fully loaded. Though there be only one carried, however, this one will be dropped if the current be at all slackened. It is clear, therefore, that though a stream have a load far less than it is capable of bearing, yet the coarser particles of that load may be dropped upon a very slight reduction of velocity.

It has been attempted to solve this question by actual experiment. Measures have been made of the sediment carried in suspension by the Upper Mississippi, the joint Upper Mississippi and Missouri at St. Louis and the Mississippi proper at Fulton, at Columbus and at Carrollton, near New Orleans.* By a careful selection of your facts you can prove, from these data, pretty much what you please. To an impartial observer, the experiments seem inconclusive. It is well known that at high stages, certain stations are places of erosion while at low stages they are places of deposition. It would depend then upon the relative duration of high and low water whether the river would be likely to be undercharged or overcharged with sediment at any given station. The experiments show with sufficient certainty, that the relation of quantity of sediment to velocity is, in individual cases, not at all a regular one, but, on the contrary, is extremely variable and apparently capricious. It seems that at the same place a velocity of 7 feet per second carried only 200 grains of solid matter to the cubic foot of water while a velocity of 3 feet carried 600. The very same velocity carried, at one time 85 grains to the cubic foot of water, and at another 2260. These, of course, are extreme cases. Doubtless by taking means the discrepancies could be greatly reduced. With the crudeness of the determinations and the many sources of error inherent in the process, and the scantiness of the data, it would be a loss of time to undertake such a task. To afford any promise of success, observations of sediment should be prosecuted at many places and for many years, with the utmost care, assiduity, judgment and good faith.

On the whole, and without reference to individual cases, it seems conformable to reason that a sedimentary stream should carry the normal burden proportioned to its mean velocity, at the different stages to which it is subject.

* See also Dr. Branner's experiments on the sediment of the Arkansas River in the "Wilder Quarter Century Book" pp. 325 *et seq.* Ithaca, 1893.

The sediment which is held in suspension by rivers is not uniformly distributed throughout the cross-section, whether from shore to shore or from top to bottom. It has been shown by Professor James Thomson that water moving around a circular curve under the action of gravity only takes a motion like that in a free vortex. Its velocity, in a channel of uniform depth, would be greater, parallel to the axis of the stream, at the inner than at the outer side of the bend. In existing streams, the section is never of uniform depth, but the side next the concave bank is deep while that next the bar, or convex bank, is shallow. The slope in the latter situation, however, is very steep, while next the concave bank it is very flat. Thus, at flood-time, the horizontal component of the velocity, or that parallel to the axis of the stream, may be as great at moderate depths on the inside as on the outside of the curve. The vertical component of the velocity, however, will be much greater next the concave bank, from the intense erosive action going on there, whereby violent inequalities of the bottom are created, and "boils" of great turbulency generated. Hence the proportion of sediment which the water holds will probably be several times greater in the one situation than in the other.

In a vertical sense, observation shows, almost without exception, that the quantity of solid matter suspended in the water near the surface is much less than the mean. The proportions, as observed at Fulton, on the Mississippi, in 1880, were, in the grand mean, for the surface 27.5 per cent., for mid depth 36 per cent., and for the bottom 36.5 per cent. At Carroliton, for the same year, the percentages were 23.4, 33.7 and 42.9 respectively. At Carrollton, in 1851-2, the proportions were 28.2, 35.1 and 36.7. In the Irrawaddy, Mr. Gordon found the proportions to be 18.4, 22.3 and 59.3. In the delta of the Rhone, M. Surrell found the proportion of surface to bottom silt about as 34.7 to 65.3. From these facts important conclusions are sometimes drawn.

It has long been known that there are two different ways in which solid matters may undergo a change of position under the influence of moving water. A man having a small weight to transfer from place to place, will pick it up and carry it. If the weight be great, he will roll it, without taking it from the ground; and there may be a less exertion of strength in the moving of the heavy load than of the light. The distance to which it will be carried in the same time, however, will be less. So it is with water. If the burden of sediment be fine clay, it will be carried in suspension. If it be heavy gravel, it will be rolled, unless the velocity be very great, as in the case of a mountain torrent. If the sediment be fine gravel or sand, it will be suspended or rolled or

both according to the fineness of the particles and the strength of the current.

It is found by experience that sediment in rivers is moved in both these ways. Where the current is insufficient to erode any matter, it may yet push it. It is observed that the solids thus transported are very much less in amount than those which are carried in suspension, but they are yet very considerable, and these movements have a great influence upon the river from their persistency and their continuity, and from the weight and inerodability of the masses transported. It is evident that a comparativly feeble current, which could not attack a bottom or bank in a vigorous way, may effect changes which shall be very perceptible after a lapse of time.

As to the exact proportion which the rolled material bears to the total quantity transported, eminent engineers differ. From the observations of Mr. Hider it is inferred by him that about 11 per cent. of the whole passes in this way. Captain Leach thinks the percentage far less—only about 1 or 2. Humphreys and Abbot estimate it at about 10 per cent. Though the quantity may be insignificant in comparison with the gigantic movements of the waterborne silt, yet by this agency, in 1883, it was found that a gravel-bar had been moved down-stream half-a-mile in four years; and the shape of the bed given by the rolling action has many important effects on the regimen of the river. The movements of sand-waves have been observed in many streams; but nowhere so closely or so accurately as in the Mississippi.

The shape of the bottom of the Mississippi, as ascertained by innumerable experiments, is that of a series of waves, of irregular shape and size, but whose irregularities mostly depend on understood causes. They do not extend directly across the river, but obliquely to the axis of the stream. The slope of the waves is always very long on the up-stream side of the crest, and comparatively steep on the down-stream side. They resemble the waves found on a sand-bar after the recession of a flood, though the latter afford but a miniature representation of the formations found in great depths and in rapid currents. The material composing the waves, which is sometimes quite heavy, is slowly rolled along the flat up-stream slope until it reaches the crest of the wave, when it falls down the steep declivity on the lower side, and there remains, to be covered, in its turn, by the material which succeeds it in its onward march. The steep slope of the down-stream face of the wave is probably due to overfall.

The conclusions reached by Mr. Hider,* after his careful observation and study of the phenomena of sand-waves are:

"1. That the bottom of the river is in an unstable condition and constantly in a state of motion. The amount of material thus moving along the bottom in waves depends on the velocity of the current and the depth of water, being the greatest when from any cause the velocity of the current is suddenly increased, at which time the most rapid erosion of the bottom takes place, and conversely the movement of material along the bottom is least when the velocity of the current is suddenly decreased, as then the greatest deposit of sedimentary matter occurs.

"2. That the bottom of the river consists of a series of ridges irregular in shape, transverse to the direction of the current, which in deep water and the most rapid current, under favorable conditions, become more regular in shape and size, approaching the form of waves. That these waves have an irregular motion down stream, and that the maximum size and rate of progression is attained when the river is at its highest stage and nearly stationary, their height, length and rate of motion being dependent on their location with reference to their distance from the thread of the main current. The waves have the least dimensions and slowest rate of travel at low water.

"3. That the heavier material makes its way down stream by being pushed or rolled up the flat anterior slope of the wave by the action of the current, and is dropped over the crest of the wave, where it remains until the wave has progressed far enough down stream to leave it again exposed to the action of the current, to be again rolled or pushed forward. The amount of heavier material thus moving is greatest in high water, or when the velocity of the current from any cause has been suddenly accelerated.

"4. That changes in the form of the waves are gradual, the waves retaining their form and individuality so long as the velocity of the current remains uniform; that they are destroyed by a rapid increase of velocity by erosion, and are obliterated by deposition of sediment, when the velocity is suddenly decreased. They again make their appearance when the velocity of the current approaches uniformity for any length of time.

The dimensions of the waves, as found by Mr. Hider, in shallow sections and at high water, were sometimes very great. The height was as much as 22 feet, the average from 8 to 18 feet. The length between

* Report of the Mississippi River Commission for 1882, p. 88.

crests bore a certain relation to the height, namely as about 30 or 40 to 1. At low water the dimensions were materially less. The rate of travel of the waves was greatest at high water, being then about 22 feet a day. At low water it was about 10.

In deep sections, the average height, at flood-time, was from 5 to 9 feet, the length 40 or 50 times the height. At low water, the dimensions were not much less than at high. The rate of travel, however, was hardly one-fourth as great—10 feet against 41.

The observations of Mr. Powless, at a point higher up-stream, differ materially from those of Mr. Hider. The average height of the waves, at all stages and velocities, was about 4.5 feet; extreme height 13 feet; average daily motion 17 feet; average length between crests about 100 times the height.

Through the instrumentality of these two agencies, solid matters are abstracted from some portions of a river and accretions are made in other portions. The principles which govern these movements may be briefly summed up in the following axiomatic statement.

If a sedimentary stream be loaded to the full capacity permitted by its velocity, and if the velocity be increased, it will acquire more sediment; if the velocity be diminished, it will drop some of the load which it already has. Matter is rolled along the bottom at all stages and conditions.

As has been intimated, the proposition that a sedimentary river is always loaded to its full capacity is probably true, if true at all, only in the grand mean, and the expression must be understood of the *permanent* capacity to carry its load an indefinite distance. It must also be remembered that even if a stream be undercharged, it will be likely to deposit some of its coarser particles if the velocity be reduced.

The practical consequence which ensues from this principle is that the beds of sedimentary streams are profoundly unstable. Every alternation of velocity from place to place brings about a translation of solid matter corresponding to the magnitude and duration of the change. Every variation of stage from high to low or from low to high produces altered conditions of cross-section and of velocity, absolute and relative, to which the channel quickly responds. Every modification of regimen, by escape of water over the banks in time of flood, by return-flow across points of bends or at the foot of lateral basins or by the influx of tributaries, introduces important changes in velocity and also in the load of sediment to be borne. Such changes occur more or less in all streams—for in none can the bed be said to be absolutely stable; but in rivers whose channel is composed of friable material, they are

so great as to give an entirely different constitution to the stream, and cause sedimentary rivers to form a class by themselves.

The changes are principally in two directions; the shifting of the whole stream laterally, by erosion on the one side and accretion on the other; and the modification of the channel vertically, by the formation of bars and pools, and the transference of material from the one to the other.

These movements are habitual, occurring from year to year with unfailing punctuality. There are others which are also of great importance, but are confined to flood-stages or to extraordinary occasions. The first class of changes leads to the development of bends and the eventual formation of cut-offs; the second to the periodical variations of regimen consequent on the transition from high to low water or the reverse, whereby great modifications are wrought in the navigable conditions of the river. Each of these subjects is of great magnitude, and would require an elaborate paper for its discussion. With regard to the first, it may merely be said that from Grand Gulf to Fort Adams, on the Mississippi, a distance of 119 miles, a comparison of surveys made in 1828 and in 1883 shows that for more than half of the above reach the present river is entirely outside of the limits of the old bed, and in some cases it is several miles from it.* As to the second class of disturbances, it may be noted that every year there are changes in the cross-section at the different discharge stations varying from 20,000 to 30,000 square feet, or to the amount of 10 or 15 per cent. of the whole area. These enormous variations of area are caused by the transfer of an incalculable volume of matter from bar to pool and from pool to bar during the transition from stage to stage.

As an example of the great practical importance of a thorough understanding of the laws governing the suspension of sediment, we may consider, for a moment, two very serious questions connected with the improvement of that great silt-bearing river, the Mississippi.

The alluvial lands of the Mississippi comprise about 30,000 square miles, all of which is more or less liable to overflow. For the protection or reclamation of this great tract, several means have been proposed. The plan adopted is that of artificial embankments or levees. As an auxiliary to this device, it has been proposed to reduce the flood-heights by waste-weirs or outlets. This plan is usually opposed by the engineers of the Mississippi service, both National and local, but their antagonism seems incomprehensible to many persons. Even among

* Mr. J. A. Ockerson, in Report of the Miss. River Commission for 1883, p. 2687.

the profession of engineers, their action is sometimes attributed to local bias or the pressure of public opinion in the communities in which they reside. Take a tub full of water, it is said, and make a hole in its side; will not the height of the water in the vessel be reduced thereby? Then why should not a hole in a system of levees reduce the floods of the river? It is acknowledged that there is a relation, more or less definite, between the height of a river and its volume of discharge. Reduce the discharge, then, by 10 or 20 per cent., and why will not you reduce the height?

It is not denied that the abstraction of a quantity of water from a river would reduce the height for the time; but it is asserted that this remedy is but temporary, and that a deterioration of the bed below the outlet would speedily ensue, and would nullify the effect of the latter. This result would be due to the reduction of velocity below the waste-weir consequent on the loss of volume of water, and also partly on the deflection of the thread of the current through the lateral vent. The effect of the decrease of velocity would be the loss of a part of the transporting power, and hence a deposit of sediment and a deterioration of the channel. The correctness of this reasoning seems to be confirmed by observation. In Holland, outlets were used for half a century, but it was found that they were absolutely detrimental, and they have all been closed. In stretches of the Mississippi River where large volumes of water have habitually escaped over the banks in time of flood, the bed of the river has actually risen to keep pace with the reduction of the discharge, so that the profile of the bottom of the river is convex in such situations.

To take another example. It is often asserted that the building of levees causes a deposit to take place in the bed of the stream. On theoretical grounds it is seen at once that this is impossible. By confining a stream we augment the volume of water without diminishing the slope. Indeed we increase the general slope; for we raise the water-surface in the upper reaches, while the sea-level is stationary. We therefore increase the velocity, and with it, in a duplicate ratio, the transporting power. So far from encouraging deposition, therefore, the confinement of flood-waters tends to produce a scour. This reasoning too is confirmed by fact. During the time since accurate observation began, on the Mississippi, there is no evidence whatever of a rise of the bed in the portion which is protected by levees, though there has been such a rise in the unprotected parts. There is, indeed, evidence that there is a decided tendency to scour, as many levee-engineers have found to their cost, in the way of caving banks.

It cannot be confidently affirmed that the building of levees will altogether prevent a deterioration of the river-bed, in all situations. It may be that the volume of silt poured into a stream by tributaries of great slope, draining a loose and erodable soil, will be more than the main river can possibly carry to the sea, especially if its lower course be through a flat plain. Such is probably the case with the Hoang-Ho, or Yellow River, of China, which flows through hills of loess or light loam, with a very steep slope, receiving several powerful tributaries, which bring down the washings of the loose and bare hillsides, and then, at its outlet from the uplands, debouches into the Great Plain, and is there turned loose to find or make a bed for itself. Too little is known of this river to venture on any positive statement regarding it, but it is possible that the tendency of such a stream to deposit a part of its load may be too great to be entirely overcome even by giving it an increase of velocity. In other words, it would fill up its bed, levees or no levees. The latter, however, would have a disposition to mitigate the evil. If the river were left to itself, the process of deposit would no doubt, in the course of time, raise the upper end of the flood-plain, and give it so great a slope, that the river would be enabled to bear its whole burden to the sea.

DESCRIPTION OF A BREAK ON THE ERIE CANAL.

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On the morning of June 5, 1895, a large break occurred in the Erie Canal at Pattersonville, ten miles west of Schenectady. The water surface of the canal at this place is twenty-three feet above low water surface in the Mohawk River and the bank which forms the side of the canal next to the river was washed away for a distance of 130 feet. Most of the water in the canal between locks number twenty-five and twenty-six poured out into the river and one canal boat was drawn into the break and totally wrecked. The distance between the locks mentioned is 6.39 miles and the canal level holds when full about 29,000,000 cubic feet of water.

The bank began to fail about 7 A. M. and most of the damage was done before 2:30 P. M., at which time the water had lowered to a depth of about eighteen inches on the easterly side of the crevasse and to a depth of about three feet on its westerly side. The water at that time was still pouring into the river at a very rapid rate.

During the forenoon the waste-gates at Sansai Kill Aqueduct, about one mile east, had been opened and some of the water was drawn off there, but it is probable that three-fourths of the contents of the level were emptied into the river through the break between the hours of 7 A. M. and 2 P. M. The rate of discharge was about 860 cubic feet per second. A better idea of the rate of discharge can be obtained by comparing it to that of the Mohawk River.

The river has a drainage area of about 3493 square miles and during an average dry season its mean rate of discharge at its mouth near Cohoes is about 1050 cubic feet per second. The discharge through the break was therefore about four-fifths that of the river.

This comparison will also help to form an idea of the extent of damage that might be done by such a stream flowing through a clay bank and having a descent of about 20 feet vertically to 110 feet horizontally.

At about 2:30 P. M. the State Engineer accompanied by others from his department arrived on the scene and proceeded to investigate the conditions surrounding the disaster and to determine upon some means of restoring the bank.

The break occurred at one end of a stone arch culvert and probably, forty feet of the culvert was washed away. This culvert had been in a leaky condition for years but had never been reported dangerous. The first indications of the break was the formation of a whirlpool in the water near the towing path and directly over the culvert. The whirlpool gradually increased in size and the towing path settled by degrees until finally it dropped into the stream and was swept away.

It seems certain that some defect in the culvert was the immediate cause of the break and it is interesting here to note the construction of such culverts as shown in the recorded plans. It must be remembered that all such structures were completed about fifty years ago and it is not possible now to learn just what plan was used for this particular culvert. The plans on file appear to be general, each plan having been used for several similar structures. *Fig. 1* shows the general section of one of these culverts along its axis. It will be observed that the top of the arch is about three feet below the canal bottom and that on one end is shown a cut-off wall of masonry which extends, from the plank foundation up the sides and across the arch forming a continuous shoulder around the masonry. Many of the culverts were built with this shoulder which was designed apparently to check any possible flow of water along the wall. On the other end of the figure it will be noticed that this shoulder has been omitted. Many other culverts were built in this manner without the cut-off. It was the usual practice to fill over the culverts with puddled clay and it might seem probable that when this was done no water would ever find a passage along the culvert, but puddling is not always well done and even if it were the frosts of fifty winters may well have rendered it somewhat porous. It is the custom to draw the water out of the canal in winter and the bottom and sides are not protected from frost.

There was nothing left of the broken end of this culvert from which to learn whether or not the masonry cut-off had been used in this case but it seems a plausible theory that the cause of the break was the absence of this safe guard and that a small stream had finally worked its way through the bank along the arch and so caused the break.

It is intended to put a cast iron pipe under the canal at this place as a substitute for the masonry culvert and when the pipe is laid it can be determined whether these cut-offs were omitted. The other end of the

culvert remains intact and can be examined when excavating to lay the pipe. Before the arrival of the State Engineer, the boat had broken in two and the lumber was being unloaded. The stern of the canal boat was resting on the bottom of the canal, its bow on the top of the culvert and its center rested in the bottom of the washout. The boat was loaded with pine boards one inch thick. The water was still running around its stern in a rapid stream and the canal bottom was steadily yielding to its action.

The plan decided upon for the repairs was to drive rows of sheet piling transversely to the excavated channel and about six feet apart, the tops to be cut off level and to furnish a support for a wooden trunk to serve as a temporary culvert.

Under the towing path two rows of sheet piling were to be fitted around and over the trunk and extended up to the canal water surface and the space between these rows to be filled with puddle. The sheet piling was to be triple lap and made up of the lumber taken from the wrecked boat. The earth for filling and restoring the bank and canal bottom was to be taken from a field about 2000 feet distant. A map of the washout showing the position of the sheet piling as constructed is shown in *Fig. 2*.

This plan having been decided upon the first step was to shut off the remainder of the water by building coffer dams across the canal. This was not accomplished until about 11 P. M., and before that time the bank had caved back so far that the stern of the canal boat had dropped down in the washout on a level with the broken part.

A force of men had been put to work unloading the lumber early in the afternoon and as many men were worked as possible, but the lumber was not all out until 5:30 P. M., of the next day, June 6th.

During the night of June 5th a temporary bridge over the canal was begun and at 10 A. M., June 6th, it was completed. Teams immediately began hauling earth over the bridge and dumping it on the berme bank and in the bottom of the washout.

Word had been sent to the surrounding villages that help was needed and by noon as many men and teams had arrived as could work advantageously. More men and teams continued to arrive and work was carried on continuously, day and night until the canal was ready for traffic.

The bow of the wrecked boat was cut off and rolled about twenty-five feet away with tackle. The stern was then cut off, the sides broken down so as to lie flat on the bottom and the stern rolled over on the sides and away from the boat which laid next to the wreck. It should

be stated that this wrecked boat was the first of a triple header, all three being loaded with lumber and shackled together. Although it was intended to burn the wreck it was necessary to be very careful to avoid setting fire to the other boats. After all due precaution had been taken the fire was started and by the morning of June 7th, very little of the boat was left. The remains were the bottom and a part of the sides and these were buried under the earth filling. The light of the fire made the night work easy and so on the following nights large bonfires were kept burning continuously.

The West Shore Railroad parallels the canal at that place and arrangements were made to have several train loads of gravel dumped there for use in making puddle. Clay alone is sometimes used but the gravel when used freely gives greater weight and stability, is not so apt to slip or crack and resists erosion by running water where clay alone would easily wash away. As the new bank would be about twenty-five feet high and needed to be perfectly safe to hold water immediately after completion it was thought best to use the gravel.

The earth used for filling was so light that it could not be packed well under the remaining portion of the old culvert around the piles. It was therefore decided to mix the gravel dry with the earth in equal parts. This mixture was thoroughly rammed and packed under the culvert. The joints of the masonry, both inside and outside, were thoroughly cleaned and pointed with Rosendale cement mortar mixed two to one. In driving the sheet piling so much of the wreck was encountered that only those rows immediately under the towing path were made perfectly tight. Trenches were dug into the banks on the sides of the crevasse and these two rows of piling and the puddle between were extended well into the banks to avoid the possibility of a crack opening at the junction of the new work with the old bank.

The rows of piling were not driven as near together as planned because it took so much time to drive them that the other work was being delayed. Every effort was made to push the work as rapidly as possible and whenever it was found that one part of the work was likely to delay the whole, all available help was concentrated on that part to push it along and avoid delay.

Fig. 3 shows the wooden trunk and the manner of its construction. The bottom was made of three thicknesses of boards laid lengthwise of the trunk and across the rows of sheet piling. The sides were made of boards piled up flatwise three and one-half feet high, and the top consisted of one tier of three inch maple plank laid transversely and two tiers of one inch pine boards. Where the trunk passed through the

two main rows of sheet piling the piling was carefully fitted to the trunk to leave no place through which the water might find a passage. The parts of the trunk were all thoroughly spiked together and as care was taken that all joints should be broken, the structure was very strong. It will doubtless be remarked that the foundation for the trunk was very poor and such in fact was the case, but it must not be forgotten that the canal was wholly obstructed and the problem was not so much how to build a perfect structure as it was to put the canal in safe condition to be used for the remainder of the season in the shortest possible time.

The plan adopted for the construction of the trunk rendered it very flexible and it could and did settle considerably without suffering any injury save to the floor. It was a mistake to run all of the floor plank lengthwise of the trunk for in settling, the pressure underneath forced the floor up and it had to be braced down afterwards as shown in *Fig. 4*. If some of the floor planks had been laid transversely this difficulty would not have arisen.

The puddle between the main sheet piling was composed of approximately one part gravel and two parts clay. The gravel was scattered over the whole surface in wheel-barrow loads and spread out with shovels; the clay was cast in from the sides with shovels. The mass was kept about as wet as lime mortar and it was thoroughly cut with shovels. It became necessary to excavate a little of the puddle after it had stood twenty-four hours and it had become so hard that to loosen it with a pick was very difficult.

The whole embankment was wet occasionally to hasten its settlement. Water was led to the work in troughs from the canal beyond the nearest coffer dam and after the work had risen higher than the water a diaphragm pump was used. When the sheet piling had risen a little higher than canal bottom it was decided to carry the puddle up the remainder of the height without it and to narrow up the puddle wall. This was done partly because the water pressure was so slight that the puddle alone was considered sufficient to withstand it, and partly to enable the teams to drive over the bank more freely and so render it more compact.

The junction of the timber trunk with the old masonry was closed up with cement mortar and a few rubble stone. After that was done and the bank had risen a little above canal bottom the lower coffer dam was cut and the water let on to the work to test it. A leak in the culvert was soon discovered and an examination showed that the rubble masonry at the junction was loosened and the mortar washed away.

A further examination showed that most of the pointing done on the masonry had hardly set at all and it proved later to be entirely worthless.

Whether this was due to poor cement or to improper mixing of the mortar was never determined. The fact that it set better in some places than in others seemed to indicate that it had not been well mixed. Another difficulty was discovered in the settling of the timber trunk. The end next to the masonry rested on some of the old piles and had not settled at all but the settlement near the centre of the bank caused the top joint at the junction to open and doubtless was the first cause of the leak. The joint had not opened an inch and the water was immediately shut off. The joint was uncovered and a jacket of quick setting Portland cement concrete was built around the junction. This jacket was very successful, it being so strong that although the further settlement of the trunk caused slight cracks in the masonry, the jacket was not injured. This extra work delayed the completion of repairs about twenty-four hours. After the concrete was in place both coffer dams were opened and the water let on the work. This time everything appeared to be in good condition and when the bank had been carried up to its full height orders were given to begin feeding the level. This was at midnight on the night of June 11. All rubbish had been removed from the canal prism; one of the remaining boats laid in a bad position and had been lightened and everything was in readiness for navigation. At 4 A. M., June 13, boats began to pass through the level. The total delay had been about eight days. After the water was in, gravel was brought on the State scow and deposited on the slopes of the prism and on the towing-path.

On the night of June 14, another leak occurred in the culvert which for a time threatened to be serious. The floor plank under the masonry culvert had been originally laid lengthwise of the culvert and the water having permeated the whole of the new filling forced up some of the plank. This was wholly unexpected and caused much anxiety until the leak was stopped. Some baled hay was secured and forced down to the leak and covered with gravel. That stopped the leak and the planks were then relaid and braced down in the same manner as had been done in the new trunk. A screen was placed over the entrance of the culvert to avoid its becoming clogged with rubbish.

The water continued to leak through the old masonry all summer and a watch was kept at the culvert continuously. The amount of this leakage was small, it being about equal to a stream two feet wide, one inch deep and having an inclination of one in thirty.

A large quantity of gravel was boated to the place and deposited over the culvert and that choked up the cracks and diminished the leaks.

The filling which had been used was so light and fine that it settled not less than three feet in the bottom of the canal and much of it was washed away in the leaks. Enough of the gravel was therefore used to restore the canal bottom to its original elevation.

There were used in the repairs about 3800 cubic yards of material; 2600 cubic yards were hauled about 2000 feet; 500 cubic yards were wheeled in with barrows, the average wheel being about 110 feet. About 700 cubic yards of the material were puddled. The sheet piling and timber trunk required about 27000 feet B. M. of lumber. The time from beginning work to the commencement of feeding was seven days and twelve hours.

It is a remarkable fact and one worthy of record that no drunkenness nor disorder of any kind occurred during the work.

It is interesting to observe that while clay is invaluable for making water-tight walks when well protected against wash, it is worthless to stop leaks in running water; and gravel alone, though very porous, will choke up cracks and crevices and stop leaks where other means fail.

Another interesting fact worthy of being remembered is that although the filling under the culvert had been carefully and thoroughly done with dry materials, yet when the materials became saturated with water, the pressure due to the head above was so little diminished that the planks were forced up from the bottom of the culvert.

In designing a new stone culvert for such a location, it would be well to use the cut-off walls already described and to lay the floor plank transversely to the axis of the culvert. The latter change would increase the resistance to the flow of water but a slight increase in the size of the culvert would readily compensate for that.

THE PROBLEM OF THE TIDES, AND THE LIMITATIONS OF THE PRESENT SOLUTION OF THAT PROBLEM.

JOHN F. HAYFORD, C.E.

The problem of the tides is an exceedingly complicated and difficult one. Any serious, extended treatment of it involves intricate mathematical processes which have exacted much hard labor from those who have attempted the solution,—and some of these attempts have been made by the most able mathematicians of their time. The theory involved, borders on the one hand on the theory of wave motion in liquids, and on the other on the exceedingly complex theory of the moon's motion. In dealing with the tidal problem, or problems, there is no lack of facts,—there is a great mass of well determined facts,—known quantities,—to be dealt with,—for the tide has been carefully observed at thousands of stations and a mere recapitulation of the salient facts might easily be made to occupy the whole time alloted to this lecture. Moreover, the main problem of the tides when treated thoroughly, gives rise to numerous auxiliary problems touching upon many different matters, as for example, the improvement of harbors, the accurate determination of mean sea level, the past history of the earth and its moon, the determination of the mass of the moon, the rigidity of the earth as a whole, the invariability of our unit of time, the recently discovered variation of latitude,—and others.

Enough has been said to show that any treatment of this subject as a whole in an hour and a half, must be limited to a bird's eye view only. I shall be quite content if I succeed in showing the subject as a whole in its true perspective, even though hazy in its outlines.

The progress which has already been made in knowledge of the tidal facts, and in constructing an adequate tidal theory, may be indicated by two contrasts.

When the Romans invaded Britain, even though they were at the time the best informed people on the face of the earth, they were taken unawares by the ten or fifteen-foot tides of the English Channel and were

surprised into the temporary loss of some of their boats which were floated off by the rising tide. Though they doubtless knew of the small Mediterranean tide, such a rise and fall as this was to them unheard of and mysterious.

Today the general character and approximate range of the tide is known over the whole world and the present day navigator need never be surprised by the tide anywhere. This volume which I hold before you, the Tide Tables of the Coast and Geodetic Survey, contains *accurate predictions* of the time and height of every high and low water which will occur at each of seventy stations in the year 1896. These stations are scattered along the coasts of every continent. In addition, the Tables give approximate predictions for three thousand stations scattered over the whole known world though, of course, most numerous on our own shores. Similar predictions are published by other governments. Provided with such printed predictions, the navigator of today encounters tides which are as resistless as of old but which are no longer mysterious or unexpected.

On the trip from Puget Sound to Sitka, Alaska, by the usual inside route among the islands which line the coast, there are two points at which the channel becomes a narrow passage, studded with sunken rocks, through which the tidal current rushes with a velocity of ten or more miles per hour during all but a few minutes of each day. To attempt either of these passages at any other time than near slack water is to invite speedy destruction. If one has access to the pilot house of a large steamer on this trip he will find that several hours before reaching the dangerous narrows the captain consults the printed predictions of tides and currents, locates his vessel upon the chart, and modifies her speed so as to arrive at the narrows at the time when the tidal current is just changing in direction and is therefore weak. The steamer reaches the hazardous narrows and encounters nothing more formidable than an irregular but moderate current, where a few minutes before would have been found a gigantic mill-race in which the vessel would have been doomed. The average passenger may perhaps wonder at the slowness of the vessel, or her unusual speed, during the hours when the dangerous point is being approached, he may perhaps notice the swirling of the currents in the narrow passage, but the triumph of tidal prediction has been so complete that he does not even suspect that any especial danger has been avoided.

Now contrast the ancient *theories* with those of today. In the olden times the mysterious rise and fall of the sea was attributed to the breathing of the monsters of the deep, or the caprice of the gods. Now

the causes of the tides are definitely known and the theory of their method of action is sufficiently exact to enable the predictions which have been mentioned to be made. With the present tidal theory it is even possible to compute the mass of the moon with a fair degree of accuracy from the records furnished by automatic tide gauges.

Still, it must carefully be borne in mind that even the present tidal theory is very incomplete and is subject to embarrassing limitations.

The word tide, is from the Anglo-Saxon word *Tid*, meaning time. This derivation limits the use of the word tide to periodic motions which bear a fixed relation to time,—in other words to periodic variations which recur at regular intervals of time. And the word, from its derivation, includes all such regular periodic motions regardless of what their causes may be. But the word tide cannot properly be applied to irregular oscillations or temporary disturbances of the sea brought about by transient meteorological causes. To speak of the piling up of the waters on a lee shore in a gale as a tidal wave, is to use "newspaper English."

The word tide covers of course both the horizontal and the vertical motion of the water, the current, as well as the rise and fall of the water surface. But for convenience in this paper, the word tide will be limited to the rise and fall of the surface, and the concurrent horizontal motions will be referred to as tidal currents. This limitation of the word is quite common in the technical literature of this subject.

In dealing with an especially difficult and complicated problem it is often advantageous for the sake of clearness of conception, to substitute for the real problem a much simpler ideal problem from which most of the complications have been omitted. Then having dealt with this simple problem, to introduce one by one the various modifying circumstances, and gradually transform by successive approximations the original simple ideal problem into the actual problem of nature. That is the method which will be used in this paper. And the order in which the complications are introduced will be in the main the historical order of their introduction in the great progressive study of the tide which has been carried on in the past.

As the fundamental simple ideal problem let us inquire as to the shape of the free ocean surface if the earth were made up of a rigid nucleus covered with a considerable uniform depth of water over its whole surface, and if it were moving in a circular orbit about the sun under the influence of its attraction, and keeping always the same face toward the sun,—just as the moon now always presents the same face to us. The necessary deviating force to keep the earth in its circular

path would be $\frac{M_e V^2}{r}$ in which M_e is the mass of the earth, V is its linear velocity along its orbit, and r is the radius of the orbit. Or, if we substitute for the linear velocity V, ωr, in which ω is the angular velocity of motion in the orbit, the necessary deviating force becomes $M_e \omega^2 r$. But the actual deviating force is, for this case, the attraction of gravitation of the sun upon the earth or $M_e M_s \frac{1}{r^2}$ in which M_s is the mass of the sun. For the earth as a whole the two expressions are necessarily equal, if the earth is to continue in a circular orbit, or

$$M_e \omega^2 r = M_e M_s \frac{1}{r^2} \quad . \quad . \quad . \quad (1)$$

Whence, for the earth as a whole

$$\omega^2 r = M_s \frac{1}{r^2} \quad . \quad . \quad . \quad (2)$$

in which r is, strictly, the distance from the center of the sun to the center of the earth.

For a small mass M_1 forming a portion of the earth and situated *at its center*, we have from (2)

$$M_1 \omega^2 r = M_1 M_s \frac{1}{r^2}$$

or, expressed in words, the direct action of the sun upon this particular mass is just sufficient to keep it in its circular orbit without any constraint from the surrounding portions of the earth.

Suppose now a small mass M_2 be considered which forms a portion of the earth at a point a distance r_2 nearer the sun than the center. Under the assumed condition of the earth moving in its orbit with the same face always toward the sun this particular small mass will be moving with an angular velocity ω in a circular orbit having a radius $r - r_2$. The necessary deviating force to keep this mass in this path will be $M_2 \omega^2 (r - r_2)$, while the actual attraction of the sun for it will be $M_2 M_s \frac{1}{(r-r_2)^2}$.

By reference to (2) it is evident that

$$M_2 \omega^2 (r - r_2) < M_2 M_s \frac{1}{(r-r_2)^2} \quad . \quad . \quad (3)$$

or that the actual attraction of the sun on this particular mass is more than sufficient to keep it in its orbit. The mass M_2 will then tend to move in a path of smaller radius and approach the sun unless constrained by surrounding portions of the earth's mass. Portions of the

rigid nucleus would be so constrained. But one hemisphere of the envelope of water,—the one nearest the sun,—would in all parts tend to move off toward the sun relative to the solid nucleus. A tangential motion having a component toward the sun would actually take place at every point of this hemisphere until the waters had piled up toward the sun to such an extent that the slope of the surface furnished the constraint equivalent to the difference of the two members of (3).

Now consider a small mass M_2 forming a part of the earth and situated in the hemisphere remote from the sun. Let its distance from the sun be $r + r_2$. By the same reasoning as before, from (2) as a basis, it will be found that

$$M_2 \omega^2 (r + r_2) > M_2 M_s \frac{1}{(r+r_2)^2} . \quad . \quad . \quad (4)$$

In words (4) means that the actual attraction of the sun upon M_2 is not sufficient to keep it in the path which it must follow if it is to remain fixed in its position relative to the earth's center. Stresses will therefore be set up in the solid nucleus, and the waters of this remote hemisphere will pile up toward a point diametrically opposite the sun, until the slope of the water surface furnishes the constraint at each point represented by the difference of the two members of (4).

For this simple case, the original spherical surface of the water would be changed to an ellipsoid with its longer axis in the line joining the earth and sun, that is, a high water would be formed opposite the sun as well as toward it. The exact form of this ellipsoid can be determined analytically, although the process is a difficult one involving complications which do not appear on a first examination. This simple tide may be called the static or equilibrium tide.

As a second approximation to the conditions of the actual tides of nature, suppose the earth to have its present uniform rotation on its axis, the conditions otherwise being as before. Ignore for the time being the effects of the inertia of moving water, of the viscosity of the water, and of the friction of the water against the bottom. The result would be that the static tide as outlined above would now move round the earth continually with unchanged form and size with its two high waters always toward and opposite to the sun.

Reasoning similar to the above would also hold good for the moon as a tide producing body. The action of the moon upon the earth would produce a lunar static tide similar in form to the solar static tide but having, on account of the nearness of the moon, a height a little more than twice as great. Neglecting as before, the effects of inertia, viscosity

and friction, this lunar static tide would follow the motions of the moon so as to keep its two opposite high waters on the line of centers of the moon and earth.

Under the conditions so far imposed, the tide upon the earth would be made up of two separate waves, each progressing regularly westward around the earth, one following the sun and the other the moon. The resultant tide would in effect be a wave following the moon,—on account of the greater range of the lunar tide,—but modified by the smaller solar tide.

If, still considering the depth of the water to be uniform and great, the effects of inertia, viscosity, and friction of the water against the bottom be now introduced into the problem, it becomes much more difficult, but still is a solvable problem. The form of the mathematical solution would now be given by the theory of wave motions in liquids. The combined effects of inertia, viscosity, and friction would be to reduce the height of the wave and to cause its highest point to lag a certain distance behind the moon, but the wave would still be of the same general character as the static tide described above.

Let us now proceed to a consideration of those actual conditions which in nature modify greatly the westward progress of this wave.

One of the laws of wave motion in liquids which will greatly influence the problem in hand,—unless the depths be very great at all points of the ocean,—is that whenever the lengths from crest to succeeding crest of a series of waves is not small, as compared with the depth of the water through which the wave is traveling, the rate of progress depends directly upon the depth according to the law $V=\sqrt{gh}$ in which $V=$ velocity of propogation, $g=$ acceleration due to gravity, and $h=$ depth of water.

According to this law the tidal wave will succeed in keeping pace with the moon in its apparent progress around the earth only in case the uniform depth is greater than thirteen miles.

If the depth of the water is uniform but less than thirteen miles the tidal wave produced by the moon wili not be propogated westward fast enough to keep up with the moon. It will tend to lag farther and farther behind. After it has lagged a certain distance the tendency will be for the direct action of the moon to build up a new wave in advance of the old one. What takes place will be in effect that the old wave will continually be being lost and a new wave continually be being built up in advance of it. The wave resulting from this action would be a forced wave, still following the moon regularly around the earth, but at an increased distance behind and having a height reduced still more than before. The problem is still tractable though exceedingly difficult.

Now let the probiem become nearer like that of Nature by supposing the shape of the surface of a rigid portion of the earth to be just what it is in fact. Suppose all the present irregularities of surface to exist,—great continental elevated areas, great irregular oceanic basins, mountain ranges, broad valleys, great plateaus, etc. But let the problem still differ from that of Nature by supposing that the water level is just high enough to cover the top of Mt. Everest, so that the whole earth would be completely covered with depths varying from zero at Mt. Everest to over ten miles on some few small areas. If now an attempt be made to determine the westward progress of the wave, the problem will at once be found to be intractable.

In the first place, the depths being everywhere less than thirteen miles, the wave will nowhere be able to keep pace with the moon. From the law stated above, namely $V = \sqrt{gh}$, the wave will move forward at about fourteen miles per minute where the depth is ten miles, and less than five miles per minute where the depth is not greater than one mile.

Keeping these rates of propogation in mind let an attempt be made to trace the progress of a tidal wave westward, starting from the Pacific Ocean. The northern portion of the wave would soon encounter the eastern portions of submerged Asia and would begin to move at the comparatively slow speed due to reduced depth, while the middle and southern portions of the wave would sweep westward rapidly in great depths across the Southern Pacific and over the bed of the Indian Ocean. By the time the middle portion of the wave had reached the eastern slope of submerged Africa, it would be far in advance of the northern part which had been struggling over the shoals of submerged Asia. And the front of the wave must have become very irregular on account of the various rates of propogation of the separate portions which had encountered different depths. As the wave progressed still farther westward the middle would find itself in shoals over the submerged plateaus of Africa, the northern part would still be struggling across Asia while the southern end would move rapidly westward in deep water south of the Cape of Good Hope. This southern portion rounding the Cape of Good Hope would necessarily be propogated northward as well as westward, for the middle and northern parts would not yet have reached the Atlantic. This Cape wave as it progressed northward up the Atlantic valley would combine at an angle in some complex fashion with the irregular wave emerging gradually from the shoals of Africa, and finally from Europe. In short, even with the conditions so far imposed it will be impossible to compute the wave to be found in the Atlantic, to say nothing of the additional complexity produced when this already complicated wave crossed the submerged Americas.

But variations in depth produce still other effects than those considered. The increased friction in small depths tends to reduce the height of the wave. On the other hand, when a wave progresses from deep into shallower water there is a tendency for the wave to attain a greater height because nearly the same amount of kinetic energy must be concentrated in a much smaller amount of water. So a tidal wave continually varies in amplitude as well as in its rate of propogation, in a way that is exceedingly difficult,—almost impossible,—to compute. The tidal wave which we have been considering in passing from the Indian Ocean across submerged Africa would probably at first increase in range on account of the concentration of energy, but afterward would, it is likely, be decreased so much by friction that it would be overpowered in the Atlantic by the tide which had passed around the Cape of Good Hope. It is evident that these continual variations in range still further complicate the already seemingly hopeless problem of tidal progress.

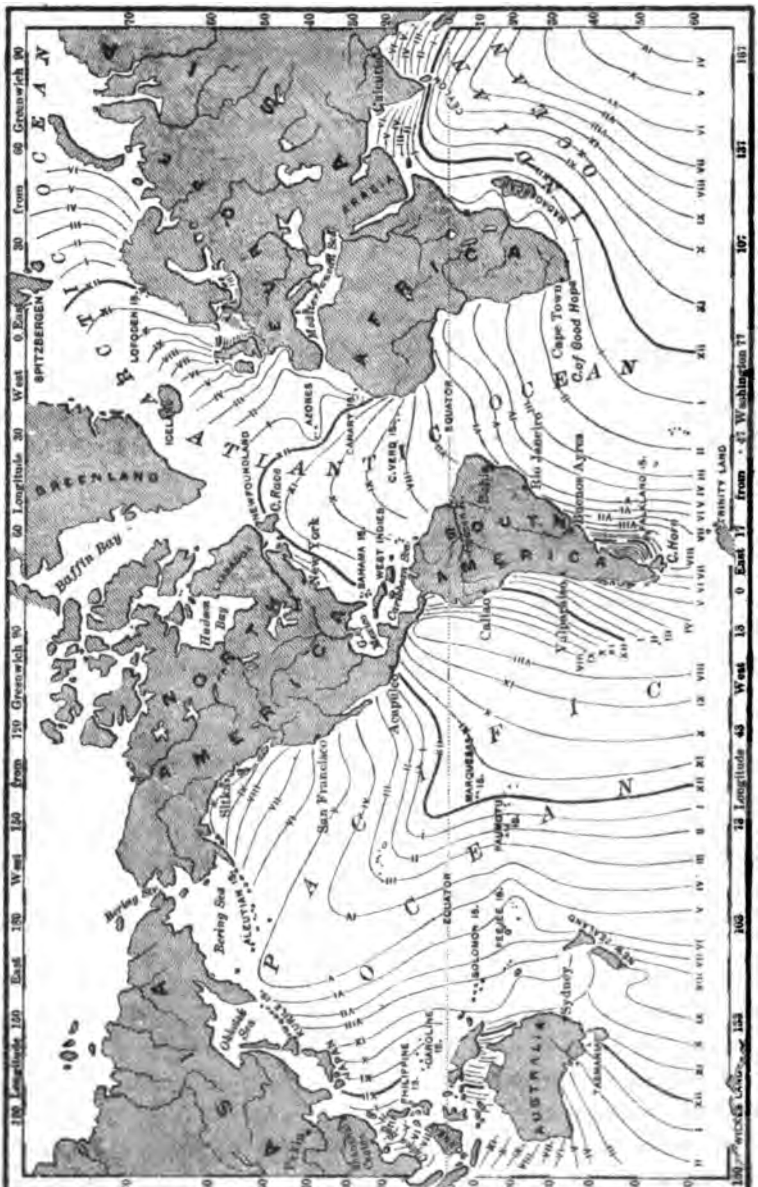
Now let the water surface which has been supposed to be at the level of the summit of Mt. Everest subside to the present position. One fourth the earth's surface becomes dry land. There is now in the problem all the previous intricacies and in addition a new set of difficulties arising from the fact that the oceans are bounded by irregular shores from which the tidal wave is *reflected* to a greater or less extent at every contact. The actual tidal waves may more properly be compared to a choppy sea, such as may be seen among the docks of a crowded water front, than to a regularly progressive double wave passing to westward around the earth, as it is usually pictured.

In short when all the complexities are considered it is evident that the problem of the *progress* of the tide is one which our limitations effectually prevent us from solving.

The map on the opposite page reproduced from Prof. C. A. Young's General Astronomy, (and originally from Guyot) indicates the observed progress of the tide in the ocean. The co-tidal lines are "drawn upon the surface of the ocean connecting those places which have their high water at the same moment of Greenwich time. They mark the crest of the tide wave for each hour of Greenwich time."

Note that according to this map a certain wave which starts off Callao, South America, at eight o'clock, divides into a westward progressing wave and an eastward bound wave. The wave which goes westward, takes about sixteen hours to reach Australia and the coast of Asia, and requires still twelve hours more to cross the Indian Ocean to Africa. In *twenty-nine* hours from the time it left its starting point off Callao it has only progressed as far as the Cape of Good Hope and lacks a

MAP OF CO-TIDAL LINES.



From Young's General Astronomy.

FIG. 156. — Map of Cotidal Lines.

Ginn & Co., Publishers. By permission.

good deal of having circumnavigated the globe. After nearly a day and a half of westward progression it collides off Cape Horn with the eastward progressing wave which started off Callao a whole day later than the westward bound wave which has been under consideration. The westward bound wave,—modified to a certain extent by the Cape Horn collision,—now proceeds northward up the Atlantic, finally swinging around to an eastwardly course in the Arctic seas north-east of Iceland.

Note that according to this map the progress of the tide is almost directly *northward* in all the Atlantic and a considerable portion of the north Pacific,—about one-third of the whole ocean surface; that the progress is actually to the *eastward* over two large areas,—west of South America and in the Arctic ocean; that even the portions of the wave which moves in the deeper and least obstructed parts of the oceans fails decidedly to make the circuit of the earth in twenty-four hours; and finally that instead of only two crests, or high waters, being in existence on the ocean surface at any instant there are at least *six* crests in existence at any moment. Compare this state of affairs with the customary elementary conception of a double tidal wave following the moon in its apparent westward progression. The customary conception is not even a rough approximation to the truth.

In looking over this map it should be borne in mind that our knowledge of the tide in the open ocean is very limited indeed. Almost all stations of observation are on the coasts. The positions of the co-tidal lines near the coasts are therefore pretty well known, but remote portions are quite largely a matter of conjecture. This uncertainty may be illustrated by the fact that Prof. Ferrel, probably the best authority on tides in this country, suggested that it is likely that the motion of the tide in the deep water of the North Atlantic is largely an *eastward* and *westward* swinging motion, like water in a wash-bowl, rather than the northward progression indicated by the map.*

So far, the tides in the open ocean only have been considered. If tides at coasts, in gulfs, bays, groups, and rivers be considered, much more marked complexities are found.

In general, when a tide encounters a great deal of frictional resistance, as in a shallow bay or in a river, the front slope of the wave becomes steeper than the rear slope, like a wind wave running up a sand beach. In rivers the front of the wave may even become vertical in some rare cases and the wave breaks and forms the so-called bore of the Seine and other rivers. In its progress up a river the range of the

*Tidal Researches pp. 238–239.

tide in general becomes gradually smaller,—though it may temporarily increase from concentration of energy—and finally the tide disappears, either because its kinetic energy has all been gradually lost in friction, or because it encounters a shallow rapids, at which the remaining kinetic energy is promptly dissipated in a futile attempt to make headway against a current which is more rapid than the progress of the wave. The tide often ascends rivers far beyond the limit of salt water and disappears finally in fresh water. In extreme cases, such as the La Plata and Amazon, the tide does not disappear until it has reached a point as much as *a hundred feet* above sea level.*

Perhaps the best example of concentration of energy in the tide is furnished by the Bay of Fundy,—where the extreme range is seventy feet and over.

Lynn Canal, Alaska, is a nearly straight deep channel running northward for a hundred miles inland from a point near Sitka. From the known depths, it would be expected that the tide would run from the mouth of the Canal to the north end in about an hour. As a matter of observation it is known that the high water occurs at the upper end about the same time or even perhaps a little sooner than at the lower end. Stating the matter as a paradox,—the tide traverses the hundred mile Canal in zero or negative time. This is probably an effect due to reflection from the abrupt end.

The difference in time of high water at Governor's Island, New York Harbor, and at the western end of Long Island Sound is about three hours,—the Sound tide being the later. These two waves, one of which has gone around the eastern end of Long Island into the Sound, and the other has progressed up the East River from New York harbor, meet at Hell Gate and produce such a complicated tide that there are points within one mile of each other at which the time of high water differs by a whole hour.

These few illustrations, from among the many that could be given, serve to indicate how great an influence the boundary conditions have upon the tide after it has once touched the shore line of a continent.

If one takes a broad view of the whole matter of the progress of the tidal wave both in the open ocean and after touching the shore; considers how complicated is the effect of bottom and of boundary upon the rate of progress, range, and shape of the wave; and compares the actually observed effects of the boundaries and bottom with such effects as computed according to the most satisfactory theories extant; one is

*Young's General Astronomy, p. 288.

forced to realize that at present the limitations of our knowledge prevent our dealing with this part of the tidal problem, except in a feeble and inadequate way.

Being compelled by Nature to admit incompetency when we attempt to trace the effect of boundary conditions upon the progress of a tidal wave, let us look upon the matter from a different point of view. Let our efforts be limited to the problem of predicting the tide at a given point, having reliable observations *at that same point.* In efforts along this line a large measure of success has been attained.

If the tides were caused by a single tide-producing body, the moon, if the apparent motion of the moon were uniform in right ascension and it remained in the plane of the equator at a constant distance from the earth, the problem of predicting the tides at any station at which observations were available would be a simple one. For, however complicated might be the progress of the tide, however greatly it might be modified by reflection and friction, however choppy the motion of the water surface might become, it would necessarily be true that the tide *at any one point* would be one of two complete waves per lunar day, with a constant range, and with each wave of exactly the same shape as any other at that point. Having once determined by observation the shape of the curve at a given station, the epochs of the lunar day at which the two high waters occur, and the range of the tide, to predict for a given future time one would simply carry forward this wave by complete lunar days to the date in question.

But the moon is not confined to the plane of the equator. In less than fourteen days it may pass from a declination of nearly 30° North to nearly 30° South, and back again in the next thirteen and a half days. Then, too, the distance of the moon from the earth changes through a range of 10% with a period of about twenty-seven days. Moreover, the interval from meridian transit to meridian transit of the moon may be anything from 24 h. 40 m., to 25 h. Each of these variations of the moon's motion must produce a corresponding variation in the tide at every station.

The sun acts as a second tide producing body standing in a continually changing position relative to the moon and therefore producing ever changing modifications of the controlling lunar tide. The sun is itself subject to changes in declination, distance, and apparent motion in right ascension similar to those of the moon although relatively much smaller.

The tide then *at any one station* is one which is constantly varying because of the varying motions of moon and sun, and their continually

changing relative position. How shall these variations be taken into account for prediction purposes? How may predictions be made?

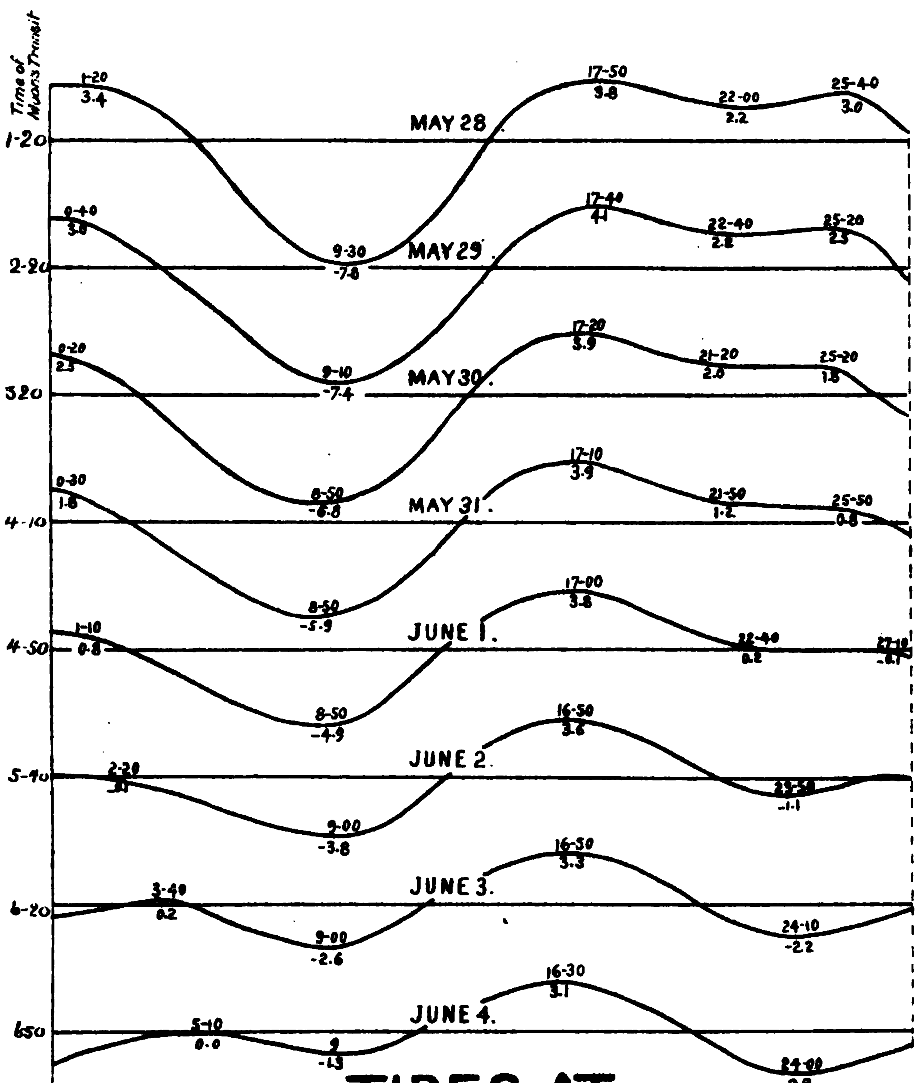
Following somewhat the historical order a first approximate prediction may be made by ignoring the *variation* in the tide, assuming it to be produced entirely by an imaginary mean moon with a uniform motion in the plane of the equator,—and deriving from the observations the mean luni-tidal interval for high and for low water, and the mean range. By luni-tidal interval for high water is meant the interval of time from the transit of the moon at the station to the occurrence of high water at that station,—and similarly for low water.

As a second approximation, prediction on this basis may be made more accurate by deriving further, from the observations, a system of corrections to the luni-tidal intervals and heights depending upon the phase of the moon,—in other words, upon the relative position of sun and moon. Such predictions could be made at any future time, the positions of the sun and moon being known in advance from published ephemerides or computations.

These predictions being found to be quite rough their accuracy may be increased by deriving from the observations additional tables of corrections depending on the declination of the moon, or the declination of the sun, on the distance of the moon, and so on. But this method quickly becomes very cumbersome, gives only approximate results for many stations, and fails utterly for those stations (and they are not few) at which the tidal wave has only one high water per day at times, instead of two.

The Port Townsend, Wash., tides as shown on the opposite page furnish a good example of such a case. The curves represent the predicted tides for May 28–June 4, 1896. Note that on May 30, May 31, and June 1 but *one* high water and *one* low water occur in twenty-four hours, and that on May 28 and again on the 29th one of the low waters occurs *above mean sea level.* The days for which the curves are drawn are not exceptional. Such phenomena as are shown occur every month. The predicted tides were used because a record of observed tides was not at hand when this lecture was being prepared. But the predictions are known to be so accurate that they serve as well for illustration as would the observed tide, in so far as the present purpose is concerned.

The rolls from an automatic tide guage at Port Townsend for 1874, show that in 130 cases during that year high water was either missing or did not differ from the preceding low water by more than ten inches, the interval between being practically a stand. The tides of all the



TIDES AT
PORT TOWNSEND WASH.
MAY 28—JUNE 4, 1896

The straight line through each curve represents mean sea level. The figures on the curves are intervals after moon's transit and heights above mean sea level in feet. The times on left margin are those of moon's transit.

Pacific coast of North America partake to a considerable degree of the peculiar characteristics of the Port Townsend tide. The tides of the Gulf of Mexico are also of this same general character,—at Galveston there being much of the time but a single high water in twenty-four hours.

It having been found that the methods of prediction indicated above were unsatisfactory, the harmonic method of analysis was applied to the problem.

In applying this analysis it is assumed that the height of the water surface at any time may be represented by the formula

$$h = A_1 \cos(\alpha_1 t + \beta_1) + A_2 \cos(\alpha_2 t + \beta_2) + A_3 \cos(\alpha_3 t + \beta_3)$$

in which h is the height of the water, referred to near sea level.

A_1, A_2, ... are the amplitudes, or semi-ranges of the various component tides.

β_1, β_2, ... are the values of the angles at the origin of time (when $t = o$)

And α_1, α_2, α_3 . . . . . are the rates of change of the various angles.

α_1, α_2, α_3 . . . . . are determined from theory only. A_1, A_2, A_3 . . . . . and β_1, β_2, β_3 . . . . . are constants to be derived from the observations.

Each term $A \cos(\alpha t + \beta)$ represents a simple harmonic motion and it is thus assumed that the actual tide may be regarded as compounded of a number of component tides of various periods, each one of which is a simple harmonic motion.

The component tide due to the mean moon is represented by a term $A_1 \cos(\alpha_1 t + \beta_1)$ in which α_1 is given a value $\frac{(2)(360°)}{\text{mean lunar day}}$, —this component thus having two high waters in each lunar day. Similarly the principal solar tide is represented by a term $A_2 \cos(\alpha_2 t + \beta_2)$ in which α is 30° per mean solar hour,—this tide therefore having two high waters per solar day.

The method of fixing the values of the remaining α's is too complicated for treatment here. Suffice it to say, that each α, and therefore the period of each component tide, is fixed from theory so that for every variation in the tide-producing forces, due to a periodic variation in the position of either tide-producing body (the sun or moon) there is introduced into the formula a harmonic term of the corresponding period.

Having fixed upon the form of the function and the values of the various α's, the values of the β's and A's are computed from the observations, by the method of least squares. The computation is quite

ingenious and much simpler than the form of the function would seem to indicate. But time forbids a description of the process here.

The number of harmonic terms indicated by theory is quite large. But many of the terms are of such small amplitude that they may be neglected. It is found that for making the most accurate predictions *nineteen* of these terms are usually sufficient.

Aside from theory simply, the assurance that the quantities A_1, A_2, A_3 . . . . and β_1, β_2, β_3, etc., of the formula are really constants as assumed, rests upon the fact that when a long series of observations at a station is treated, the values for the constants *as derived from the computation* are found to be practically the same, regardless of whether the computation covers the whole series, or any single year of it. Some series comprising as much as nineteen years of continuous automatic record, have been computed by single years and the constant character of the quantities A_1, A_2, ... β_1, β_2, ... demonstrated by the close agreement of the results year by year.

Both theory and direct experiment agree thus in asserting that the harmonic analysis gives the invariable elements of the tidal motion.

The harmonic method is at present the usual one throughout the world for computing tides for the purpose of accurate prediction. The Tide Tables for 1896 as published by the U. S. Coast and Geodetic Survey, give the harmonic constants of the tide for seventy stations scattered over the whole world.

The following table gives the amplitudes of the harmonic components for a few stations which are selected as being typical. Diurnal components are indicated by a subscript 1 and semi-diurnal by a subscript 2. It may seem strange that there should be a component tide, K_1, with sidereal speed—as if the stars produced a tide. K_1 is due to the effects of the declination of the moon and the sun and has no connection with the stars. Similarly, K_2 has nothing to do with stars.

The non-harmonic methods are used for subsidary computations and for series of observations which are too short for the effective use of the harmonic method.

Having now the nineteen (say) components of the tide at a station, and these components being known to be invariable, to make a prediction for any future time it is simply necessary to carry each component forward by means of its known period to the time in question. But as soon as one attempts this process it is fonnd to be very slow. It takes one computer about six weeks to predict the time and height of each high and low water for a single year at a station, using nineteen components. One is thus in about the position of the boy who took his father's clock

AMPLITUDES (OR SEMI-RANGES) OF THE COMPONENT TIDES IN FT.

Designation of Component.	Sandy Hook, N. J.	San Diego, Cal.	Port Townsend, Wash.	Honolulu, Hawaiian Islands.	Yokohama Entrance, Japan.	Calcutta, India.	Liverpool, Eng.	Speed of component, a, in degrees per solar hour.	REMARKS.
K_1	0.33	1.01	2.47	0.48	0.73	0.39	0.36	15.041069	One high water per *sidereal* day.
K_2	0.11	0.20	0.16	0.04	0.16	0.45	0.94	30.082138	Two high waters per *sidereal* day.
L_2	0.12	0.05	0.11	0.03	0.04	0.20	0.53	29.528478	Gains on M_2 as fast as N_2 loses on it.
M_1	. .	0.06	0.08	. .	. .	0.03	0.03	14.492052	An octave lower than M_2,—so to speak.
M_2	2.19	1.70	2.24	0.52	1.18	3.62	9.99	28.984104	Two high waters per lunar day. The principal lunar tide.
M_4	0.03	0.03	0.13	. .	. .	0.72	0.69	57.968208	An octave higher than M_2,—so to speak.
M_6	0.05	0.01	0.03	0.00	. .	0.16	0.20	86.952312	Three times as fast as M_2,—a harmonic tone,—so to speak.
N_2	0.52	0.41	0.47	0.09	0.24	0.65	1.91	28.439730	Loses on M_2 as fast as L_2 gains on it.
O_1	0.17	0.70	1.41	0.26	0.60	0.21	0.37	13.943036	Loses on M_1 as fast as K_1 gains on it.
P_1	0.11	0.35	0.77	0.14	0.24	0.14	0.13	14.958932	
Q_1	0 03	. .	0.26	0 04	0.12	0.03	. .	13.398661	
S_2	0.43	0.70	0.55	0.17	0.59	1.48	3.17	30.000000	Two high waters per solar day. The principal solar tide.
T_2	. .	. .	0.08	. .	. .	0.15	. .	29 958932	
λ_2	. .	. .	0.03	. .	. .	0.89	0.23	29.455626	Gains on M_2 as fast as ν_2 loses on it.
μ_2	0.07	0.03	0.08	. .	. .	0.24	0.26	27.768208	Loses on M_2 as fast as S_2 gains on it.
ν_2	0.12	. .	0.09	0.02	0.05	0.25	0 53	28.512582	Loses on M_2 as fast as λ_2 gains on it.
MS_4	. .	. .	0.07	. .	. .	0.65	0.41	58.984104	Gains two whole waves on M_2 in each solar day.
S_a	0.14	0.23	0.22	0.22	0.30	2.71	0.39	Annual component,—one high water per year,—due to meteorological causes.	

to pieces. He knew all about the separate parts, but the clock would not now tell time for him. So far we have succeeded in separating the tide into its component parts but can not expeditiously put those parts together again.

Here computing machines become necessary to make extended predictions feasible. And, fortunately, the formula in question is one which lends itself very readily to mechanical computation.

The machine used by the English for tidal predictions combines mechanically the separate cosine curves and draws on a roll of paper the composite curve, which represents the tide for that station at a definite future time. The predicted times and heights are then read off from this curve precisely as if it were a curve of observation drawn by an automatic tide guage.

In this country the predictions have been made by the use of the Ferrel Tide Predictor at the Coast and Geodetic Survey Office. This machine having been set for a certain station from the computed harmonic constants at that station, a single operator can predict the tides for a year in a single day. The operator manipulates the machine by a single crank with his left hand, reading times and heights of high and low water directly from the face of the machine, and writing with the right hand. The rapidity of work is limited practically by the rate at which the operator can copy correctly from the face of the machine.

The Tide Tables above referred to for 1896 give predictions made in this way for every day of the year for seventy stations. The following sample pages, 46a and 46b, reprinted from these tables will serve to show the present form of publication of these predictions. A careful study of these pages will also furnish a contrast in the character of the Pacific and Atlantic coast tides of the United States.

To form an idea of the errors of these predictions, the *predicted* tides for the year 1889 at Sandy Hook were compared with the actual tides for the same time and place, as shown by an automatic tide guage.* It was found that nearly fifty per cent. of the predicted times of high and low water agreed within fifteen minutes with the observed times, and that sixty per cent. of the predicted heights agreed with the observed within 0.45 ft. To properly appreciate these differences it should be kept in mind that they depend upon the errors of the original observations from which the constants were computed, the errors, if any, made in computing those constants, the small errors due to yielding, etc., of the machine Predictor, the errors in the observations of 1889 with which the predictions were compared, and finally and largely, the non-predictable disturbances of sea level by meteorological causes. The available evidence indicates strongly that the larger differences are due in the main to this last cause. In regard to the time differences, it should be noted that the water often stands sensibly at the same level for as much as fifteen minutes and therefore an error of fifteen minutes is practically about as good as zero error. A single discrepancy in height of 3.7 feet occurred on Sept. 10 during a great storm.

The Coast Survey is at present engaged in building a new tide predictor to replace the Ferrel machine which has rendered such good service for many years. The new tide predictor will do at the same time both the work of the English machine and the Ferrel Predictor. That is, it will not only draw the tidal curve for any future time as the

*See C. and G. S. Report 1890, appendix No. 15.

SANDY HOOK (New York Entrance), NEW JERSEY, 1896.

OCTOBER.

Moon.	Day of— W.	Day of— Mo.	Time and Height of High and Low Water.			
	Th	1	1:40 3.6	7:46 1.0	14:07 4.3	20:46 0.6
	F	2	2:52 3.9	9:02 0.7	15:15 4.5	21:47 0.3
	S	3	3:57 4.3	10:00 0.3	16:26 4.8	22:43 −0.1
	S	4	4:54 4.8	11:03 −0.1	17:17 5.1	23:34 −0.4
E	M	5	5:45 5.2	12:08 −0.4	18:10 5.2	. . .
●	Tu	6	0:24 −0.6	6:35 5.5	12:54 −0.7	19:01 5.3
P	W	7	1:10 −0.8	7:20 5.8	13:45 −0.9	19:51 5.3
	Th	8	1:56 −0.8	8:10 5.9	14:35 −0.9	20:40 5.1
	F	9	2:44 −0.6	8:56 5.8	15:20 −0.7	21:30 4.8
	S	10	3:31 −0.4	9:46 5.6	16:17 −0.5	22:24 4.5
S	**S**	11	4:24 −0.1	10:40 5.3	17:13 −0.2	23:19 4.2
	M	12	5:19 0.2	11:36 5.0	18:18 0.1	. . .
☽	Tu	13	0:20 3.9	6:22 0.5	12:36 4.6	19:16 0.3
	W	14	1:20 3.8	7:30 0.7	13:42 4.4	20:19 0.3
	Th	15	2:22 3.8	8:30 0.8	14:46 4.2	21:13 0.4
	F	16	3:00 4.0	9:41 0.7	15:43 4.2	22:10 0.4
	S	17	4:06 4.2	10:37 0.4	16:41 4.2	22:57 0.3
E	**S**	18	5:12 4.4	11:26 0.4	17:27 4.3	23:39 0.2
	M	19	5:38 4.5	12:00 0.3	18:08 4.2	. . .
	Tu	20	0:17 0.2	6:20 4.6	12:50 0.3	18:48 4.1
A ○	W	21	0:50 0.2	7:03 4.7	13:26 0.2	19:24 4.1
	Th	22	1:22 0.2	7:35 4.8	14:00 0.2	19:56 4.0
	F	23	1:51 0.2	8:06 4.8	14:34 0.3	20:30 3.9
	S	24	2:20 0.3	8:36 4.8	15:06 0.4	21:06 3.8
	S	25	2:51 0.5	9:11 4.7	15:44 0.5	21:40 3.7
N	M	26	3:30 0.7	9:50 4.7	16:26 0.6	22:24 3.6
	Tu	27	4:10 0.7	10:36 4.6	17:12 0.5	23:14 3.5
	W	28	5:06 0.9	11:28 4.4	18:00 0.5	. . .
☾	Th	29	0:14 3.6	6:11 1.0	12:30 4.3	19:10 0.4
	F	30	1:20 3.8	7:27 0.8	13:38 4.3	20:13 0.3
	S	31	2:27 4.1	8:40 0.6	14:46 4.3	21:13 0.1

NOVEMBER.

Moon.	Day of— W.	Day of— Mo.	Time and Height of High and Low Water.			
E	**S**	1	3:28 4.5	9:47 0.3	15:58 4.5	22:00 −0.2
	M	2	4:26 4.9	10:43 −0.1	16:53 4.7	23:02 −0.4
	Tu	3	5:18 5.3	11:44 −0.5	17:46 4.9	23:54 −0.6
P	W	4	6:10 5.6	12:36 −0.8	18:40 4.9	. . .
●	Th	5	0:43 −0.8	7:00 5.9	13:27 −0.9	19:31 4.6
	F	6	1:32 −0.7	7:46 5.9	14:18 −0.9	20:22 4.8
S	S	7	2:21 −0.6	8:36 5.8	15:00 −0.8	21:14 4.6
	S	8	3:11 −0.4	9:28 5.6	16:01 −0.6	22:07 4.4
	M	9	4:06 −0.1	10:20 5.3	16:55 −0.4	23:03 4.1
	Tu	10	5:00 0.2	11:12 4.9	17:50 −0.1	. . .
	W	11	0:01 4.0	6:03 0.5	12:10 4.5	18:46 0.1
☽	Th	12	1:02 3.9	7:06 0.7	13:12 4.2	19:42 0.3
	F	13	2:01 3.9	8:10 0.7	14:10 4.0	20:35 0.4
E	S	14	2:56 3.9	9:00 0.7	15:03 3.9	21:23 0.4
	S	15	3:47 4.1	10:04 0.7	16:00 3.8	22:13 0.4
	M	16	4:32 4.3	10:53 0.6	16:48 3.8	22:56 0.3
A	Tu	17	5:13 4.5	11:37 0.3	17:32 3.8	23:34 0.3
	W	18	5:50 4.6	12:20 0.2	18:12 3.8	. . .
	Th	19	0:11 0.3	6:28 4.7	13:00 0.2	18:53 3.8
○	F	20	0:45 0.3	7:03 4.8	13:36 0.2	19:33 3.8
	S	21	1:21 0.3	7:38 4.9	14:12 0.1	20:10 3.8
N	**S**	22	1:56 0.3	8:15 4.9	14:50 0.1	20:48 3.7
	M	23	2:34 0.4	8:53 4.6	15:30 0.1	21:30 3.8
	Tu	24	3:15 0.5	9:34 4.8	16:13 0.1	22:14 3.8
	W	25	4:05 0.6	10:21 4.8	16:57 0.1	23:04 3.8
	Th	26	4:58 0.7	11:12 4.5	17:43 0.2	. . .
☾	F	27	0:00 3.9	6:01 0.7	12:12 4.3	18:44 0.2
	S	28	0:56 4.1	7:00 0.6	13:14 4.2	19:41 0.1
E	**S**	29	2:00 4.3	8:18 0.5	14:20 4.1	20:40 0.0
	M	30	3:00 4.6	9:24 0.2	15:20 4.2	21:37 −0.2

DECEMBER.

Moon.	Day of— W.	Day of— Mo.	Time and Height of High and Low Water.			
	Tu	1	3:50 4.9	10:26 −0.1	16:28 4.3	22:34 −0.3
P	W	2	4:56 5.3	11:26 −0.4	17:27 4.4	23:26 −0.5
	Th	3	5:40 5.5	12:20 −0.6	18:20 4.5	. . .
●	F	4	0:20 −0.6	6:41 5.7	13:13 −0.8	19:14 4.5
S	S	5	1:12 −0.6	7:31 5.8	14:04 −0.8	20:06 4.5
	S	6	2:04 −0.5	8:21 5.7	14:54 −0.8	20:58 4.4
	M	7	2:56 −0.3	9:10 5.5	15:44 −0.7	21:50 4.3
	Tu	8	3:47 −0.1	10:00 5.2	16:33 −0.5	22:41 4.2
	W	9	4:41 0.2	10:40 4.8	17:20 −0.3	23:34 4.1
	Th	10	5:36 0.4	11:30 4.4	18:00 0.0	. . .
☽	F	11	0:25 4.0	6:36 0.7	12:32 4.1	18:56 0.2
E	S	12	1:16 4.0	7:30 0.8	13:24 3.8	19:45 0.4
	S	13	2:07 4.0	8:26 0.8	14:18 3.5	20:33 0.4
A	M	14	2:57 4.0	9:21 0.8	15:12 3.4	21:20 0.5
	Tu	15	3:45 4.1	10:13 0.7	16:02 3.4	22:06 0.5
	W	16	4:30 4.3	11:03 0.6	16:52 3.4	22:50 0.4
	Th	17	5:12 4.5	11:48 0.4	17:39 3.5	23:33 0.4
	F	18	5:54 4.7	12:30 0.2	18:24 3.6	. . .
N ○	S	19	0:15 0.3	6:36 4.8	13:11 0.1	19:08 3.7
	S	20	0:57 0.2	7:16 4.9	13:51 −0.1	19:52 3.8
	M	21	1:41 0.2	7:57 5.0	14:33 −0.2	20:34 3.9
	Tu	22	2:24 0.2	8:40 5.0	15:14 −0.3	21:16 4.0
	W	23	3:08 0.2	9:23 4.9	15:56 −0.3	22:01 4.1
	Th	24	3:56 0.3	10:10 4.8	16:40 −0.3	22:50 4.2
	F	25	4:50 0.3	11:00 4.6	17:37 −0.1	23:40 4.2
E	S	26	5:47 0.4	11:58 4.3	18:16 −0.1	. . .
☾	**S**	27	0:34 4.4	6:50 0.3	12:51 4.1	19:11 0.0
	M	28	1:34 4.5	7:57 0.3	13:56 3.9	20:10 0.0
	Tu	29	2:35 4.6	9:05 0.2	15:08 3.9	21:10 −0.1
P	W	30	3:36 4.8	10:10 0.0	16:07 3.9	22:10 −0.2
	Th	31	4:36 5.1	11:12 −0.2	17:08 4.0	23:00 −0.3

The tides are placed in the order of occurrence, with their times on the first line and heights on the second line of each day; a comparison of consecutive heights will indicate whether it is high or low water.

The time used is Eastern Standard, 75th meridian W.; 0^h is midnight, 12^h is noon; all hours less than 12 are in the morning, all greater are in the afternoon, and when diminished by 12 give the usual reckoning: for instance, 15^h is 3 p. m. The heights, in feet and tenths, are reckoned from Mean Low Water, which is the datum of soundings on the Coast and Geodetic Survey Charts for this region, and which is 2.3 feet below mean sea level. Symbols and abbreviations relating to the moon: ●, new moon; ☽, 1st quar.; ○, full moon; ☾, 3d quar.; E, moon on the equator; N, S, moon farthest north or south of the equator; A, P, moon in apogee or perigee.

PORT TOWNSEND, WASHINGTON, 1896.

APRIL.

Moon.	Day of— W.	Day of— Mo.	Time and Height of High and Low Water.			
	W	1	5:07 10.3	12:12 1.0	19:52 9.9	. . . . . .
	Th	2	0:34 7.4	5:32 9.9	13:03 1.0	21:14 9.6
S	F	3	1:14 8.2	5:54 9.5	14:00 1.2	22:36 9.6
☾	S	4	2:30 8.7	6:18 9.2	15:01 1.6	23:43 9.7
	S	5	4:00 8.8	7:30 8.5	16:08 2.1	. . . . . .
	M	6	0:47 9.9	6:00 8.1	9:28 7.8	17:04 2.7
	Tu	7	1:28 10.6	7:30 7.2	11:55 7.9	18:10 3.2
	W	8	1:54 10.6	8:30 5.5	12:15 7.9	19:04 3.8
	Th	9	2:17 9.9	8:54 5.2	14:19 8.2	19:55 4.4
E A	F	10	2:37 9.6	9:10 4.7	15:14 8.4	20:32 5.0
	S	11	2:54 9.6	9:16 3.8	15:50 8.9	21:02 5.5
●	S	12	3:12 9.7	9:40 3.1	16:31 9.2	21:32 5.9
	M	13	3:28 9.8	10:07 2.5	17:17 9.3	22:08 6.5
	Tu	14	3:42 9.9	10:37 1.8	18:08 9.2	22:38 7.0
	W	15	3:58 10.0	11:11 1.3	18:54 9.4	23:18 7.5
	Th	16	4:16 9.7	11:30 0.9	19:52 9.5	. . . . . .
N	F	17	0:08 8.1	4:34 10.0	12:33 0.8	20:54 9.6
	S	18	0:54 8.6	5:06 9.7	13:20 0.9	21:58 9.7
	S	19	2:00 8.7	5:42 9.8	14:15 1.3	22:55 10.0
☽	M	20	4:00 8.5	6:30 8.7	15:15 1.8	23:44 10.1
	Tu	21	5:30 7.9	8:58 8.2	16:18 2.4	. . . . . .
	W	22	0:31 10.1	8:27 6.7	11:00 7.7	17:28 3.0
	Th	23	0:54 10.0	7:08 5.7	12:44 8.1	18:36 3.8
E	F	24	1:24 9.7	7:38 4.4	13:54 8.8	19:33 4.6
	S	25	1:50 9.8	8:10 3.2	14:55 9.6	20:34 5.1
P	S	26	2:22 10.0	6:54 2.0	16:00 10.2	21:07 6.0
○	M	27	2:46 10.3	9:36 1.0	16:58 10.5	21:51 6.5
	Tu	28	3:12 10.3	10:18 0.2	17:50 10.3	22:36 7.5
	W	29	3:36 10.1	11:03 —0.2	19:00 10.4	23:20 8.1
S	Th	30	4:03 10.6	11:48 —0.2	20:04 10.2	. . . . . .

MAY.

Moon.	Day of— W.	Day of— Mo.	Time and Height of High and Low Water.			
	F	1	3:12 8.6	4:25 9.7	12:37 0.2	21:03 10.3
	S	2	2:00 8.8	4:36 9.2	13:28 0.8	22:08 10.3
	S	3	4:20 8.6	5:43 9.5	14:19 1.6	22:56 10.4
☾	M	4	6:30 7.6	7:48 7.4	15:12 2.5	23:36 10.2
	Tu	5	7:20 6.7	10:30 7.0	16:14 3.4	. . . . . .
	W	6	0:24 10.0	7:24 5.7	13:08 6.5	17:18 3.6
E	Th	7	0:48 9.9	7:50 4.8	13:29 7.2	18:11 5.2
A	F	8	1:08 9.7	8:06 4.0	14:28 7.7	18:54 5.9
	S	9	1:27 9.8	8:24 3.1	15:18 8.0	19:34 6.7
	S	10	1:38 9.9	8:45 2.2	16:05 8.6	20:10 6.8
	M	11	1:55 9.9	9:10 1.6	16:50 8.5	22:48 7.6
●	Tu	12	2:07 10.1	9:36 0.9	17:33 9.5	21:28 8.0
	W	13	2:28 10.3	10:07 0.3	18:14 9.7	23:13 8.3
	Th	14	2:46 10.5	10:43 —0.1	18:57 10.1	23:00 8.6
N	F	15	3:14 10.0	11:24 —0.3	19:44 10.3	. . . . . .
	S	16	0:01 8.7	3:50 9.7	12:01 —0.1	20:31 10.8
	S	17	1:06 8.7	4:36 9.5	12:54 0.6	21:18 10.9
	M	18	2:24 8.5	5:31 8.8	13:47 1.0	22:00 10.7
☽	Tu	19	3:44 7.8	7:12 7.8	14:44 2.0	22:40 10.5
	W	20	5:00 6.7	9:00 7.2	15:48 3.1	23:17 10.3
E	Th	21	5:43 5.1	11:38 7.4	16:50 4.4	23:44 10.0
	F	22	6:21 3.9	12:55 8.2	17:51 5.3	. . . . . .
	S	23	0:03 10.0	7:10 2.5	14:15 8.7	18:33 6.1
P	S	24	0:48 10.2	7:55 1.3	15:24 9.7	19:46 6.3
	M	25	1:18 10.2	8:40 0.3	16:25 10.0	20:38 7.5
○	Tu	26	1:46 10.5	9:24 —0.5	17:23 10.4	21:26 8.4
S	W	27	2:12 10.9	10:05 —0.9	18:16 10.5	22:20 8.8
	Th	28	2:42 10.2	10:43 —1.0	19:08 10.6	23:18 8.9
	F	29	3:08 9.8	11:32 —0.6	19:50 10.9	. . . . . .
	S	30	0:06 9.0	3:35 9.3	12:13 0.0	20:41 10.7
	S	31	0:41 8.8	4:40 8.6	12:55 0.9	21:28 10.7

JUNE.

Moon.	Day of— W.	Day of— Mo.	Time and Height of High and Low Water.			
	M	1	2:00 8.0	6:00 7.0	13:40 1.9	21:46 10.6
	Tu	2	3:30 7.9	8:00 6.7	14:26 3.0	22:31 10.4
☾ E	W	3	5:30 5.7	10:00 7.0	15:15 4.2	23:04 10.1
	Th	4	6:30 4.6	12:05 6.8	15:56 5.5	23:24 9.9
A	F	5	6:56 3.9	13:26 6.9	16:37 6.4	23:05 9.9
	S	6	7:21 2.9	14:34 7.4	17:30 7.0	23:52 10.0
	S	7	7:45 2.0	15:32 8.1	18:30 7.6	. . . . . .
	M	8	0:18 10.3	8:13 1.2	16:18 8.4	19:15 8.2
	Tu	9	0:33 10.2	8:40 0.6	17:00 9.1	20:43 8.7
	W	10	0:48 10.0	9:00 0.0	17:38 9.4	21:12 9.0
● N	Th	11	1:30 10.3	9:44 —0.8	18:12 10.1	21:55 9.0
	F	12	2:00 10.2	10:23 —0.8	18:44 10.5	23:00 8.9
	S	13	2:45 10.6	11:08 —0.8	19:18 10.5	23:54 8.8
	S	14	3:07 9.8	11:47 —0.5	19:54 11.1	. . . . . .
	M	15	0:55 8.1	6:40 9.0	12:38 0.4	20:30 11.1
	Tu	16	2:00 7.4	5:48 8.1	13:24 1.4	21:05 10.8
E	W	17	3:10 6.7	7:37 8.9	14:16 2.5	21:00 10.5
☽	Th	18	4:04 5.1	9:47 7.2	15:07 4.2	23:05 10.2
	F	19	4:55 3.7	11:38 7.7	16:01 5.7	22:34 10.3
P	S	20	5:54 2.4	13:14 8.1	17:06 6.3	23:12 10.4
	S	21	6:48 1.3	14:34 8.7	18:06 7.8	23:47 10.5
	M	22	7:38 0.1	15:41 9.6	19:18 8.5	. . . . . .
	Tu	23	0:30 10.2	8:27 —0.7	16:33 10.2	20:50 9.1
S ○	W	24	0:58 10.3	9:13 —1.0	17:38 10.5	21:00 9.3
	Th	25	1:45 10.1	9:56 —1.1	18:12 10.7	22:30 9.2
	F	26	2:18 9.7	10:16 —0.8	18:51 10.6	23:00 8.8
	S	27	3:07 9.1	11:15 —0.4	19:28 11.1	. . . . . .
	S	28	0:42 8.3	4:04 8.7	11:51 0.5	19:55 10.8
	M	29	1:46 7.5	5:00 7.8	12:38 1.5	20:25 10.7
	Tu	30	2:51 6.7	6:21 7.3	13:07 2.0	20:45 10.0

The tides are placed in the order of occurrence, with their times on the first line and heights on the second line of each day; a comparison of consecutive heights will indicate whether it is high or low water.

The time used is Pacific Standard, 120th meridian W. from Greenwich; 0^h is midnight, 12^h is noon; all hours less than 12 are in the morning, all greater are in the afternoon, and when diminished by 12 give the usual reckoning; for instance, 15^h is 3 p. m. The heights, in feet and tenths, are reckoned from the mean of a selected few lowest low waters, which is the datum of soundings on the Coast and Geodetic Survey Charts for this region, and corresponds with the harmonic tide plane, which is 6.7 feet below mean sea level. Symbols and abbreviations relating to the moon: ●, new moon; ☽, 1st quar.; ○, full moon; ☾, 3d quar.; E, moon on the equator; N, S, moon farthest north or south of the equator; A, P, moon in apogee or perigee.

English machine does, but will also show upon its face ready for the copyist, the time and height of every high and low water.

It is interesting to note here, that the most wonderful computing machine in existence owes its origin to the application of the harmonic analysis to the tides. I refer to the Harmonic Analyser, a computing machine, made and used by the English. It is operated by feeding into the machine the roll of paper from an automatic tide guage and by hand retracing with a point the tidal curve. When this process is finished certain indices on the machine show readings from which the harmonic constants are derived by a simple multiplication or division. The machine then performs the converse office of the Predictor. The Predictor takes the numerous component curves and combines them for a future time. But the Harmonic Analyser performs the much more difficult office of separating the observed composite tidal curve, with all its apparent irregularities, into its separate components, of which originally only the periods are known. This machine with some minor changes may be used for the determination of the elements of any observed periodic phenomena.

To resume,— with present methods predictions of a high degree of accuracy can be made for any station at which a series of good observations is available. In fact, these predictions by the harmonic method are probably about as accurate as they can ever be made by any process whatever,—in view of the comparative certainty that the larger outstanding discrepancies at present are due to meteorological disturbances. But, strictly, said predictions are *limited to the station of observation.*

When one desires to predict a tide for a neighboring station he is at once confronted by the limitations, dealt with in an earlier part of this lecture, of our knowledge of the effect of boundaries (bottom and sides) upon the progress and range of a tidal wave. In other words, one is confronted with the problem of predicting the change in the tide which takes place between two given points, and this is precisely the problem in which we encounter the most narrow limitations of success.

If the range and time of occurence were the only tidal quantities which were decidedly changed in passing from place to place it would be comparatively simple to obtain, by observing these two quantities, a means of reducing the predictions from one station to another. Indeed, the fact that the other tidal quantities change more slowly than these two, is utilized in the C. and G. S. Tide Tables to reduce from the stations for which the harmonic predictions are made, to neighboring stations at which only a few observations of time and height are available. But such an extension of the predictions introduces errors which in-

crease rapidly with the distance. For the *relative magnitude* of the separate tidal components varies greatly from place to place in a manner that is at present non-predictable, and can only be determined by direct harmonic analysis at each point. If out of the various component tides there are formed two groups, one composed of component tides which are semi-diurnal (having two high waters per day), and the other composed of those which are diurnal (one high water per day), the preceding statement may be forcibly illustrated. Atlantic tides will be found to have semi-diurnal components predominating largely over the diurnal. In the Gulf of Mexico, on the contrary, the two classes of components are either of about the same magnitude, or the diurnal class predominates, as at Galveston where frequently there is but one high water per day. For North Pacific stations the diurnal components equal or exceed the semi-diurnal. For any one region there are also continual changes from place to place in the relative magnitudes. It is the change in the relative magnitude of these two classes of components that makes the characteristic difference between the tides at Sandy Hook, N. J., and the peculiar tides of Port Townsend with the occasional single high water per day.

In dealing with the general problem of prediction the present status is; that for all stations at which a few months of continuous observations are available an accurate prediction can be made; that for all other intermediate stations predictions can be made by interpolation,—an interpolation, the exactness of which depends largely on the shortness of the space over which it extends, inasmuch as the laws of variation of the quantities concerned are not known; that for intermediate stations at which short or discontinuous series of observations are available the interpolation can be increased considerably in accuracy by non-harmonic computations of the mean range and mean luni-tidal intervals,—these quantities serving as an aid to interpolation.

It is interesting to note that just as with musical sounds there is a tendency for harmonics of the original tone to be produced, so in the tides the fretting against bottom and shores has a tendency to generate the waves of shorter period corresponding to harmonic tones,—this tendency becoming more and more decided as the friction increases. In the table of harmonic constants on page 24, M_4 and M_6 are such component tides.

For the engineer, the determination of mean sea level, the world's datum plane, is perhaps the most important of the auxiliary tidal problems. It is not necessary here to comment on the desirability of an accurate determination of that plane. How shall mean sea level

be determined at a tidal station? It is a common practice to determine it by observing the heights of high and low water for a few days and taking as the mean sea level a plane midway between mean high water and mean low water. This method gives an approximation to mean sea level which is good enough for many purposes, especially if one is dealing with an Atlantic tide, or any tide in which the diurnal components are small as compared with the semi-diurnal components. But it is well to be forewarned that for the other class of tides, in which the diurnal components are nearly as large or larger than the semi-diurnal, the above method of reduction gives only a rough approximation to the plane sought, and that the error of the approximation can not be made to disappear even by increasing indefinitely the length of series dealt with. At Port Townsend, for example, a tide similar to that shown for May 28th, with nearly equal high waters close on each side of a very high low water, occurs frequently, while the symmetrical case of nearly equal low waters on each side of a very low high water, seldom or never occurs. The asymmetry of which this is an illustration is one of the marked characteristics of this tide and makes it true that mean sea level, determined as above indicated, from high and low water readings only, may easily be in error more than a foot from observations (say) for a week, and will be in error by several inches even when the series is of indefinite length.

It is the *area* of the portion of the tidal curve which is above mean sea level that is equal to the *area* of the portion below. Mean sea level is correctly determined. then, by taking the mean of equally spaced ordinates,—say hourly ordinates.

But even when mean sea level is determined from hourly ordinates it should be kept in mind that for most stations of the world there is a tide with a period of one year, marked S_a in the table of page 45, due to meteorological causes, having a range of from four inches to a foot. Unless a correction is applied to take account of this tide, mean sea level as computed from the observations of say a month, may be in error by from two to six inches. In some rare cases this tide is much larger. At Calcutta, as the extreme case, it has a range of over five feet.

When a line of geodetic levels starts from a tidal station and the highest degree of accuracy is desired, the question arises, "is the *mean* level of the sea at that station the same as the undisturbed level would be if no tide existed at the station, or its approaches?"* Does the tide

* See Tidal Researches, by William Ferrel, pp. 202, 206, 136-137.

bank up the water to a certain extent against the land just as the wind waves do? This effect is probably small, but that it is so, has not been satisfactorily proved either theoretically or by observation.

In conclusion, let me warn you against the most common mistake made in using tidal predictions. It is in many cases easier to judge when the tidal current changes from flood to ebb or *vice versa*, than to determine when the vertical motion changes from rise to fall or *vice versa*. So the reversal of the horizontal motion being observed, it is in many cases assumed, thoughtlessly, that the time of the reversal of the vertical motion is the same, and thus the Tide Tables are accused of being in error. As a matter of fact, taking the stations of the world as a whole, at a large percentage the tidal current does not change until thirty minutes to two hours after the vertical motion has changed. In some rare cases it is even true that the flood tide runs for about three hours after high water, so that the current is running up hill half of the time, just as a pendulum bob is on the up slope half of the time. Tidal predictions usually refer to the rise and fall only, and are so stated, and should not be taken to refer to the currents,—the horizontal component of the motion.

In preparing the lecture, aside from the notes on,—and familiarity with—tides, acquired while acting as computer in the tidal Division of the Coast and Geodetic Survey, I should acknowledge especially the following sources of information.

Tidal Researches, by William Ferrell. Appendix, Coast & Geodetic Survey Report, 1874.

The Tide Gauge, Tide Predictor, and Harmonic Analyser. Sir William Thomson. Minutes of the Proceedings of the Institute of Civil Engineers; Vol. 65. (English.)

Comparison of Predicted and Observed Times and Heights of High and Low Water at Sandy Hook, N. J. during the year 1889. Alex S. Christie. C. & G. S. Report, 1890, Appendix, No. 15.

The Harmonic Analyser; *The Engineer*, Dec. 19, 1879.

The Maxima and Minima Tide Predicting Machine; William Ferrel. Appendix No. 10, C. & G. S., Report, 1883.

General Astronomy; Prof. C. A. Young.

HOLLAND'S WAR WITH THE SEA.

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There is a country where the rivers run, so to speak, above the heads of the inhabitants, where powerful cities rest below the level of the sea which surges against them, where portions of the cultivated fields are invaded by the waters and in turn freed from them, where islands have been attached to the continent by ropes of sand, and where parts of solid ground have been transformed into islands.

Such a country is the United Netherlands, more familiar to many as Holland, the name of one of its individual states. Holland, without quarries, has built magnificent buildings, and substantial cities; almost without timber she has constructed navies which have disputed the sea with the most powerful fleets.

It is not astonishing that even a sterile country should by cultivation produce grain and stock, but it is surprising that Holland should exist.

That which interests the traveler more than the local scenery, the character of the people, or the prosperity of the country, is the mystery of formation and strange destiny which is explained partly by nature and partly by human industry. Flat as a calm ocean, indented by gulfs and bays, eaten away by interior lakes, and intersected by rivers, Holland seems to have been for ages the arena of combat between land and sea.

In other countries when science seeks to unravel geologic problems it examines the testimony of the rocks and reads from monuments regarding whose structure history is silent. Human genius follows the action of forces which spent themselves anterior to man's probable entry, but in Holland all is new; the gulfs, lakes, and islands and even entire provinces have come into existence under man's observation. He has seen within historic times, sand close a river's mouth, land converted into water, and lakes dry up and disappear. The ordinary agencies of change; wind and waves, rain and flood, and the rise and fall of land have here been at work. Long after the continent of

Europe had become fixed and stable, Holland began its geographic formation and is still pursuing processes intended to hold or enlarge her boundaries. The inhabitants of a country are what the external influences make them, and its geography is a preface to its history, as well as a key to the understanding of the people's habits, genius, and institutions.

By the aid of old maps and documents, one can know what Holland was at the time when it first became known, and what changes have been wrought by the action of the waters of its rivers, the waves of the sea, and the hands of man; in short, how Holland was made. The power of the rivers one can now see in the inundations, the action of the sea in the sand dunes along the coast, and the transformations by man everywhere.

The Dutch geologists, from a careful examination of soils obtained by borings, have divided the formation of the Netherlands under the action of fresh water into three periods: anterior to the existence of the Rhine, while the Rhine was opening a passage to the sea, and since man has attempted to keep the Rhine within defined limits.

Before the birth of the Rhine, the great part of the Netherlands was a sea, limited on the German side by a rocky coast which is now seen in the Teutoburgwald. This sea left in its ancient bed bones of the mammoth, such as are found in the North Sea, and blocks of granite evidently carried by the glaciers from Norway's mountains. The glacial sea extended as far south perhaps as the Alps. The uplifting of the Ardennes enclosed a sea in the interior of Germany and shielding it from the cold winds of the north, it soon became full to overflowing from the melting ice on its southern shores. Finally the pent up waters broke through and in the bed thus formed the Rhine has since been flowing.

With the rush of water large masses of rocks were moved and hurried along until the moving force became insufficient; smaller particles were carried further, and when the sea was reached the final burdens were surrendered. The pebbles and grains of sand on which rests the soil of Guelderland, Over Yssel, and the Island of Texel, show that their primeval home was the basalt regions of the Rhine. Just as Egypt is the Nile in its present tense, so is Holland the present of the Rhine—that Rhine which rushes over the rocks at Schaffhausen, spreads out majestically before Mayence, passes in triumph under the fortress of Ehrenbreitstein, and beats in sonorous cadence at the foot of the Seven Mountains. In its course, it reflects gothic cathedrals, princely castles, fertile hills, steep rocks, famous ruins, cities, groves, and gar-

dens. But in giving up its load for the making of a state, it has forced its recipient to wall it in and watch with anxiety its tortuous march to its death in the sea.

From the neighborhood of Emmerich, before reaching the Dutch frontier, it has lost all the beauty of its banks, and flows in great lazy curves through broad monotonous flats, suggestive of approaching old age. At Millingen it runs entirely in the territory of Holland, but only a short distance until it divides. The main branch shamefully disowns its name, and throws itself into the Meuse, a river of French origin; the other branch, insulted by the name of Dannerden canal, after going nearly to Arnheim, again separates into two branches; one empties into the Zuyder Zee, the other regaining its early name though qualified as the Lower Rhine, goes as far as the village of Drustede, where it divides for the third time. One of these branches, like a fugitive from justice, changes its name and unites with the Meuse near Rotterdam; the other still clinging tenaciously to its old name, even if inflicted with the soubriquet of "curved", reaches Utrecht with difficulty, where for the fourth time it is compelled to submit to a division. One part denying its once boasted name, drags itself as far as Muiden where it unites with the Zuider Zee; the other called the Old Rhine, flows slowly to Leyden, whose streets it languidly crosses, then gathered into a canal it is carried to its death in the North Sea.

But even this rite, decent burial, is of recent origin. In the ninth century a furious western storm not only drove back the waters of the Old Rhine, but threw across its channel great mountains of sand and blocked its entrance into the sea. The river lost itself partly in the sands near the coast and partly in pools about the surrounding country. Under the reign of Louis Bonaparte a canal was opened through the dunes and the Rhine again conducted into the sea. The mouth of this canal is protected by enormous dykes and breakwaters, and the sea itself is held in check by locks or sluice gates. When the tide is high these locks are closed to prevent the waters from invading the land; when the tide falls they are opened to give passage to the waters of the Rhine which have accumulated behind them, and then three thousand cubic feet of water a minute passes out. On days when storms prevail, a concession is made to the sea, and the outer gate is left open so that the second may sustain the shock. These enormous fortifications on the sands defending the dying Rhine and the fallen city of Leyden command respect and grateful admiration.

The tendency of the rivers of Holland has been to drop sediment along their lower levels and especially at their mouths. The sea has resisted

this encroachment and in retreating has continually fought to regain the lost territory. It has thrown barriers across the river channels to make the rivers themselves destroy the land of their creation, it has buried the rich alluvial soil fathoms deep under unproductive sands, and wherever it does not build a fortress against itself, the state must accept the challenge and begin a royal battle.

Along the North Sea there are stretches, sometimes eight miles in length, along which there must be built dykes strong enough and high enough to withstand the heaviest storm and the highest sea.

The more recent type of dyke may be described as follows: It is built of earth, firmly packed on its sea face, and partly paved with dressed Norway granite or Rhine basalt blocks. Beginning at the top, the dimensions are as follows:—sixteen feet across the top,—a width great enough to accommodate the double track railroad and a path besides; then on the sea face, it inclines at an angle of thirty degrees for a distance of thirteen feet, when the slope diminishes to only one in twenty. For forty feet a sod surface is maintained but beyond that for one hundred and ten feet where the force of the storm-lashed waves beat the hardest, the surface is faced with stone. This carries the face to a point about three feet below high tide, then a flat pavement is laid out to and beyond the low water line. As seen in the picture, three rows of piles are driven in to hold the facing in place, and two other rows of larger piles with their tops protruding along the line where the waves are most aggressive.

On the inner face the slope is more rapid and protected by sod alone. The height above high water to the railroad track is eighteen feet.

The amount of labor required to construct such a line of fortifications can hardly be imagined, and the cost is well-nigh beyond conjecture. One great item of expense is the piles—all of which come from other lands—and cost to put in place four dollars each.

This defense is not an idle precaution. When the west winds drive the waters from the English Channel to meet those deflected by Norway's shores they fill up the North Sea and seek their old course across the Netherlands. Angered at man's attempt to block the way, the waves beat remorselessly against the work of his hands and dash to pieces the stanch ships entrusted to their care. Only last March, the fishermen's boats drawn up on the sands at Schevingen did not escape the sea's fury. They were hurled across the beach and crushed one against another upon the dunes beyond.

The land around being lower than the sea, becomes so saturated with moisture that it must be drained in order to be productive. This drain-

ing is primarily into ditches, but the waters of the sea being higher there can be no direct outlet, hence the water from the ditches must be raised by artificial means high enough to run through well guarded sluice gates into the sea.

This brings us to the general drainage question. In the three-fifths of the entire territory which is below sea-level, ditches form the dividing line between farms or even between fields. When these become full the water is pumped by wind-mill power into larger ditches having higher banks, and from these into another with still higher banks until a large canal with an outlet into the sea is reached. The first lifting is done by private parties, but when larger areas are interested in the prompt handling of the water it is in the hands of the government and steam-pumps are employed. Since a large number of smaller canals are emptied into each larger one, the pumping at each transfer station cannot be unlimited, for then water might be put into a canal more rapidly than it could be pumped out. Then too the final canals emptying into the sea have their discharge limited by the height of the tide at their mouths. Thus it is necessary that at each station the height of water in the canal must have a fixed and defined limit. For fixing this limit a net-work of lines of precise levels covers Holland, and gauges at every pumping station mark the elevations in terms of a common zero of elevation—that of the Amsterdam Bench Mark. Then the man in charge of each station is told that he must not pump after the water in the receiving canal has reached a certain height, and that the water in the lower canal ought to be kept below a certain level, that is, if it rises above that height there is danger of an overflow. From this it can be seen that a local engineer might be embarrassed by two conflicting orders. He may be forbidden to pour water into one canal because it is already full and know at the same time that the lower canal is dangerously full. Fortunately for him, he is not called upon to worry about this. The latter condition had been already observed by the district inspector and notices served that pumping into this lower canal should cease at once. The canals still lower might thereby be in danger of overflow, but that would cause the pumps still lower down to stop until the individual farmers would be forced to throw their wind pumps out of gear. This might cause some fields to become submerged, but the principle is observed that it is much better for the water to rise gently over a few fields than to have a large canal burst its banks and the rushing water endanger life as well as property.

This entire question of drainage, the conduct of river waters to the sea, and the protection of exposed shores, are under the direction of the

Ministry of Water Affairs. The country is divided into provinces with a director and engineers in charge, and the provinces further subdivided into districts with more elaborate supervision over each. Officers are always on the lookout at the sea-ends of the larger water ways and the height of water due to tide or storm telegraphed several times a day to all the stations at which water is poured into a particular outlet. These stations communicate any unusual disturbance in the outflow to all their feeders. It is like a general massing his troops—a delay in the advance column causes the issue of an order to one in the rear to halt, and this order is passed back to the last company.

Heavy rains in one section give to that district a sort of right of way in passing along its extra superfluity of water, and ice blockades in the early spring give notice that very soon unusual precautions will be needed.

These faithful men, trained for their important duties at the State School of Engineering at Delft, are ever on the alert. At no time can they be negligent of their duty. A sudden northwest wind may spring up and drive the waters of the Rhine and the Maas back from their mouths. If this condition is not reported overflows from feeding canals are sure to follow.

These canals, so necessary for drainage, are also most useful as a means of communication, and boats are the most common freighters. The depth and width of each canal are known, the way in which its banks are protected decides whether the swash from a propeller can be withstood and the velocity permitted is plainly given. Convenient locks enable boats to pass from the higher to the lower canals, so that nearly any place in Holland can be reached by water from every other place. Even the ditches around the farms and fields are navigable by flat boats, the hay and barley are carried by water to the barns, and the milkmaids—usually men—go by boats to milk.

At first it looks odd to see a sail apparently moving across the landscape, and only after closer acquaintance does one realize that the sail belongs to a boat whose hull is hidden behind the banks of a canal. The wind blows nearly eighty per cent of the time over the Netherlands so that sails can be used a large part of the time, and while the canals are too narrow to admit of tacking, there is usually such a variety of water-ways connecting the place of departure with the destination that a route favorable for the sail can be chosen.

It is interesting to see in the summer afternoons how the small boats laden with vegetables come out to the larger canals leading into the cities, and after being picked up by a tug boat go into the town like a chain

of barges down the Hudson. The tug drops the boats in the harbor and the owners of each seek their accustomed market place along the canals in the city.

In the smaller canals the usual means of propulsion is poling. A man walks to the front of the boat, thrusts a long pole to the bottom of the canal and placing his shoulder against the free end of it, he walks towards the stern, literally kicking the boat from under himself. When the stern is reached, the pole is withdrawn and the process repeated, alternating sides for the purposes of steering in case only one is poling. The fact that so many of the Dutch cities are provided with canals on which the heavier freight is carried reduces the use of drays and large wagons for that purpose to a minimum. Smaller parcels are carried on push carts and their utility having once been demonstrated they now find odd and varied uses. "It's a good thing, push it along", must have originated in Holland.

Where the ditches form the boundaries for the fields they are crossed by bridges. The bridges have usually a trap door running their entire width, and the raising of this door forms an efficient barricade to the cows within the field.

There is a tradition that fish have been seen in these canals aud ditches, and I have seen thousands of men and boys never women, they are too busy—sitting on the bank with poles and lines and baited hook waiting for a bite or even a glorious nibble. I have also seen as many fish, sometimes ostensibly near a fisherman lying-in-wait, but I have never seen a fish, pole and line in their supposed proper relation. The uncertainty as to what may be transpiring beneath the surface of the murky water and the possibility that at the next minute the cork might bob, have a charm even for the onlooker, and I admit that on more than one occasion I have sat down on the grass and watched with hope as well as interest. When the cork would yield to a gentle tremor, the enviable holder of the rod would say "that is a cat," and later as he raised the baitless hook and said "I told you so" he passed for a veritable Isaak Walton. Surely the Lone Fisherman was a Dutchman.

Naturally where the lifting of water is such an important matter, the most economical means of accomplishing it is a matter of supreme moment. When only small areas are involved and the amount of percolating water not great, small wind-mills operating the simple Archimedean screw suffice. The smaller wind-mills like those of the most recent type put themselves in the wind, and turn patiently as long as the breezes hold out, or until an order comes to stop. These machines

are extremely simple in construction, and the height to which water can be raised by a screw of this character is limited by the length of the tube which can be so supported as to be free from sagging. It is evident that any appreciable sagging would prevent the inner axis from turning and thereby impair its efficiency. Fortunately, in the lowest levels where such devices are employed the height to which the water must be raised seldom exceeds two or three feet.

Large volumes of water are usually lifted by paddle wheels or breast wheels, also driven by wind power, but operated by wind-mills of greater dimensions. Some of these are owned by private individuals but many belong to districts and work in the interests of entire communities. These too have a limited efficiency, but they are doing faithful service in thousands of neighborhoods. It must not be presumed that all of the wind-mills of Holland are the drivers of water-hoisting machinery, a large majority of the more pretentious ones drive mills for grinding wheat, flax-seed, or madder, sawing logs, or crushing stone. Some are very substantially built of brick and are equivalent in value to an average farm.

At the critical, one might almost say strategic, points, steam pumps are provided for fear that an emergency might arise at the time of a prolonged calm. And although now largely employed, one must not forget what was accomplished when wind alone was used as a motive power.

The Dutch name polder is applied to any area of land protected by an enclosing dyke and drained by its own system of pumps. Some of these are but slightly below the general level, and need but a light embankment; such are usually of a firm soil and, after the removal of the water, become fields. Others were originally ponds or lakes, or deposits of wet muck, which have to be enclosed by more substantial embankments, and the removal of the water in the first instance as well as subsequently, has been a serious matter. Sometimes the removal of peat forms a pond, from which the water is afterwards taken and the dried land enriched for agricultural purposes.

The polders vary in size from two or three acres to over forty thousand acres and they lie from a few inches below the level of the water without to eighteen feet below the same. The former were drained by single mills, while in the latter cases strong dykes were required and the best pumping machinery kept in operation for years. The interior of each polder is cut with ditches and canals, which conduct the drainage to the points where pumps are established to lift the water over the embankment. In summer droughts the reverse process

brings in a bountiful supply of water for irrigation purposes. In the Rhineland district there are 90,000 acres of land which would still be under water were it not for the skill, capital and energy of the doughty Dutch warriors.

North Holland has undergone great changes in its water-washed boundaries as well as in its interior character. Even going no further back than 1288 and accepting as reasonably accurate the map of that date we can trace century by century, if not year by year, the fortunes of the constant war with the waters. By 1575, the outward form had changed somewhat, while the interior had so rapidly melted away, that the feeling became general that determined efforts should be made to prevent any further wasting away. These efforts were at first precautionary, the war being wholly on the defensive.

The holding of the natural streams in check, keeping them within their proper channel, allowed some of the marshes to become dry. This gain of land whetted the people's appetite for more, and plans were soon put into execution for draining some of the shallow lakes which had now become isolated. The labor of accomplishing these tasks was enormous, and very slow when wind was relied upon, but decade by decade saw the useless lakes disappear, until now we find large areas of land and but little water.

The most important change wrought by man upon the face of Holland was the drying up of the Haarlem Lake.

This lake, or as it was called, this sea, had been formed by the joining of four smaller lakes and enlarged by frequent inundation until it attained a circumference of thirty-seven miles. The soil of its shores was very fertile and so readily dissolved by water that no prediction could be made as to which way it would further grow. With an outlet into a branch of the Zuider Zee, known as the Y, ships could enter it and pass from shore to shore. At one time fleets of seventy ships had fought upon the lake, and on more than one occasion storms had strewn its banks with wrecks. Fortunately sand dunes skirted its western shore, but for these the lake would have joined the North Sea and Holland would have become an island. The friable character of the banks and the fierce winds which ruffled its surface made it necessary that the neighboring people should always be on the defensive.

As early as 1643, a Dutch engineer Jan Adrians, surnamed Leeghwater, published in book form a detailed plan for draining the lake. At this time Holland was too much occupied with the war with Spain to undertake a work so extensive and so fraught with difficulties as this was conceded to be. The political complications which followed the

peace of 1684, and the uncertain issues of the war between England and France, caused Leeghwater's project to be forgotten.

He had proposed a surrounding dyke, with a skirting canal on the outside, and the employment of 160 wind-mill pumps in gangs of four. At that time the Rhine was closed as has been mentioned already and the ready disposal of the water from the lake made the labor somewhat more difficult. Other plans were snbmitted—all relying upon minor power—until 1819 when the government took the matter in hand and ordered a thorough discussion of the elements of the problem.

As is usual in such matters, final action was delayed partly because of the many rival plans submitted, and partly because of the great cost of the undertaking. However, the authorities were suddenly awakened to the danger coming from this foe within by the storms of the autumn of 1836. On the 9th of November, a violent west wind drove the waters of the lake into the very streets of Amsterdam. They swept over one polder after another and covered dykes, roads and even bridges. On Christmas day a fierce east wind arose and hurled the waters of the lake back again and did not rest until a part of Leyden was inundated. When the inventory of damages was taken, it was found that 100,000 acres of land had been under water, and 18,000 acres of polder completely filled. An entire year was consumed in freeing the submerged lands from water, and great losses were found to have resulted from the overflow. This was the final provocation. The challenge was accepted and the battle was to be to the death. In 1839 the States General decided to attack this enemy and placed the entire matters in the hands of a commission of thirteen members, some of whom were engineers, others landed proprietors and state counselors.

The plans adopted comprised several independent features. First of all the building of an enormous dyke entirely around the lake for a distance of nearly thirty-seven miles, and outside of this an encircling canal 131 feet wide. This canal was to serve a triple purpose; affording a channel for the navigation now excluded from the lake, a means of conduct for the waters which formerly flowed into the lake and finally to furnish an outlet for the drainage water which would have to be pumped out during and after the formation of the polder.

At the southern end it was intended to have this encircling canal connect with the Rhine. Consequently it was necessary to enlarge the sluices at Katwyk in order to allow this increased amount of water to readily escape into the North Sea.

This canal intersected the river Sparne, which could easily with the added water get beyond its banks unless assisted in the ready discharge

of its increased supply. Hence it was necessary to erect a large pumping station at Spaarndam and for a similar reason one at Halfweg. Certain polders which had the right to drain into the Haarlem Lake had to be provided for and this necessitated a pump at Gouda. This much had to be done before the first gallon could be taken from the imprisoned lake.

Three pumping stations were erected at the edge of the lake to deliver the water into the encircling canal. They were fixed at "The Cruqius," and the others at the two ends of the longitudinal axis of the lake, points which are now the termini of the principal drainage canal of the polder.

In the early work a number of difficult engineering problems were encountered. The enclosing dyke in some places crossed tongues of land which in reality were nothing more than floating beds composed of aquatic plants whose roots were so interlaced as to rest on the surface of the water. These had to be loaded down with earth taken from the canal until they rested firmly upon the bottom of the lake, and the dyke then built on top. In other places the embankment intersected ponds or creeks where the surging of the waters would soon wear away any ordinary earth-works. Here jetties were built by piling bundles of rushes one upon the top of another and for a great part of the way they were obliged to face the water side of the dyke with sand brought from the dunes to the east of the lake.

In general the earth for the embankment was taken from the canal, and this sufficed for the canal, which for a width of ninety-five feet has a depth nearly eleven feet. The dyke itself rises to a height of about eight feet above the zero at Amsterdam. It is covered with turf, and so compact did it become as soon as built that it has settled but little.

By October, 1843, the dyke and canal were practically completed but the final closing was delayed by certain complications as well as the non-arrival of machinery, until May, 1848.

Of course the choice of means for lifting the water from the lake was an important matter. Careful computation gave the amount of water in the lake as 780,000,000 tons, and to this was to be added rainfall and water of infiltration, which was estimated to amount to 40,000,000 tons per annum during the drainage and perhaps half as much more afterwards. Then too provision had to be made for unforseen emergencies and most unfavorable conditions. Wind-mills were naturally proposed, but the most advantageous arrangement of them fixed four years as the minimum time in which the work could be done. It was therefore decided to use steam-pumps and provide for the re-

moval of 40,000,000 tons per month which would require fourteen months for draining the lake.

It would be interesting to the engineer to discuss the special expedients devised and the pumping mechanism finally adopted, It was absolutely pioneer work for operations of such magnitude. English engineers were employed and the eleven pumps at each of the three stations raised at every stroke 2376 cubic feet of water, while the total output of all the pumps in twenty-four hours was more than 1,000,000 tons.

At first only one station was equipped, so fearful were the commission that the performance of the engine and pumps might not come up to expectations. It worked alone for eleven months, during which time the level of the lake was lowered only five and one-half inches.

The other two stations began in April, 1849, and in July, 1852, the lake was dry, the work having consumed thirty-nine months instead of fourteen as at first contemplated. In this time 946,075,000 tons of water had been removed, or about fifty per cent. more than originally provided for. A gridiron system of canals in the polder is provided for the interior drainage as well as for means of communication, and the level of the land of the polder is fourteen feet below the zero at Amsterdam.

This gigantic work has been thus described in detail that you may properly appreciate Dutch ingenuity and energy and listen with respect to the projects frequently made to drain the Zuyder Zee.

The completion of the drainage of the lake was celebrated by the issue of several medals; the one struck by the Government contained in Latin the inscription : "Haarlem Lake, after having for centuries assailed the surrounding fields, to enlarge itself by their destruction, conquered at last by the force of machinery, has returned to Holland its 44,280 acres of invaded land."

These acres are now occupied by about 12,000 people, and their products are the choicest of the land. In this vast plain, so recently the bottom of a navigable lake, straight roads are bordered with trees; substantial and even elegant farm-houses are seen on every hand; throughout the whole commune there are police, cemeteries, fire-engines, all the appliances of Dutch civilization, as well organized as in any of the older districts; periodical cattle markets are held; the stage coach makes its stated trips; a steamboat plies the encircling canal; grain mills are at work; and life within the polder is quite independent of that without.

The Commission is quite pardonable when, after recounting the

material benefit resulting to the State, it says: "But this is not all; we have driven forever from the bosom of our country a most dangerous enemy; we have at the same time augmented the means for defending our capital in time of war. We have conquered a province in combat without tears and without blood, where science and genius took the place of generals, and where workmen were the worthy soldiers."

There is one place of Holland soil from which even Dutch determination cannot withhold the invading waters—one battle-ground that for generations has been held by the foe. It is the Island of Marken in the Zuyder Zee.

This island, detached from the mainland in the thirteenth century, lies out of all the ordinary routes of travel and hence its inhabitants have perpetuated the quaint costumes and queer customs which prevailed at the time that their land became an island. The ground is barely above the water at high-tide, so that any unusual storm would completely sweep over such protecting dykes as the people could afford to build. With characteristic shrewdness they long ago counted the cost of such fortifications as the exposed position would necessitate and wisely concluded that the ground at stake would not justify the expenditure. They therefore dug such canals as would drain the ground under ordinary conditions and used the earth thus obtained in building hillocks on which the houses are erected.

On seven of these mounds houses are grouped, while the eighth is the silent home of the dead. The buildings which are not so favorably situated with respect to the highest point on the hill, are built on stilts, so to speak—the lower story being merely frame-work, and only the upper one occupied, with a gangway connecting it with the adjacent houses, so that in case of an overflow isolation cannot be complete.

The dress of the women is very odd. On their heads they wear an enormous white cap in the form of a mitre, ornamented with fancy needle work and tied under the chin like a helmet. From under the cap fall two long braids, while a square-cut bang stiffly waxed forms a sort of visor. The boddice of the dress is richly embroidered in oriental patterns, the skirt made of dark blue homespun is very short and full and the shoes are of wood. The costume of the men consists chiefly of a tight jacket to which are buttoned trousers which are exceedingly full about the hips and very close-fitting below the knees. A small cap, wooden shoes, and a silver pin at the neck complete the attire.

These costumes are universal on the island, and being so distinct from those of all other parts of Holland, it shows how long ago the separation from the main land took place, and how completely isolated its inhabitants have been.

There are no trees on the island, only a few sheep, and fowls, and every particle of building material must be brought from the shore. The men are engaged in fishing. They go out Sunday evening and return on the following Saturday, after having disposed of their week's catch, and purchased such articles as the home demanded.

The women educate the children, make clothing, and keep every shining article duly polished. The large majority of them never leave the island, and the only events that vary the monotonous life are marriages, births, and deaths, the passing of a steamboat, the coming of a visitor, or the elimination of the island by a tempest.

The shifting sands in the Zuyder Zee threatened sometime ago to close up the harbor of Amsterdam and the navigation of this uncertain lake out to and around the Helder seriously embarrassed the shipping interests of this important city. It was therefore decided some years ago to construct a ship canal from Amsterdam directly to the North Sea.

This great work was completed in 1870, with these a terminus at Ymuiden. The sea being higher than the water in the harbor at Amsterdam, it is necessary to have enormous locks at this end. The traffic through this canal is so great that the water let through in the locking would be a source of danger in time. The harbor of Amsterdam is shut off from the Zuyder Zee by means of dykes with an opening series of locks. Thus the waters augmented by the inlet through the North Sea Canal, the drainage from the Haarlem Lake and the Anstel river, now cut off from their natural outlet into the Zuyder Zee would soon overflow the city if it were not for the powerful pumps which lift the accumulated water ont into the lake. Thus the canal, so necessary to Amsterdam's prosperity, is in league with the enemy ; the Haarlem polder which is the city's kitchen garden, drains its superfluous water into the rising harbor, and the Zuyder Zee barred out because of its fickleness takes to itself the threatening flood, only after a great outlay for pumping.

The war with the watery element is not always open and above ground, in digging nearly every foundation a superabundance of water is encountered, and the building itself must rest upon piles. It is always fascinating to watch a pile driver, to see it swing a great log erect and into place, then stroke after stroke drive it home. I have inspected some building operations there, have seen the men, provided with high top boots, cleaning away the foundations ; a steam pump industriously striving to keep the water out and the pile driver thumping away. One morning the entire foundation was full of water and a second pump

was called into action. At last rows of piles were in place, rows like the teeth of a comb, but the pump could not stop. The tops of the piles were cut off at the same height, tenons cut on them and great horizontal beams mortised to them. The space between these beams filled with sand and the whole covered with a very heavy flooring—but the pump kept up its monotonous throbbing. On this floor the brick walls are erected and soon a great six-story building will stand on wooden feet. When will the pumping cease? Never. Under the building there is a catch basin and whenever it becomes full it must be emptied, and this I suspect will be very often, for the canal at its side is six feet higher than the basement floor.

The sea, too, had its allies, two in number, both living and though man's inferior in strength and genius they by their rapid increase are worthy foes.

The sand dunes which are thrown up along the sea coast form protecting hills, but as free sand, they shift so much and so rapidly that if unrestrained they may leave a vulnerable point unprotected and bury productive lands beneath a blighting mass. It therefore becomes necessary to check the migrations of these shifting ridges, and this is done by planting upon their sides a species of reed grass. These grow quite rapidly even in the sand, and very soon their roots—forming a species of vegetable cement—aid in holding its nourishing soil in place. But these same roots are tempting tid-bits for the burrowing rabbit, and while seeking the food made possible by man's desire to protect himself, they open insidious tunnels which, though small, are dangerous as opening wedges. It is, therefore, necessary to make war upon this ally and be always upon the alert to arrest the damages before it is too late.

The other foe is the dreaded teredo, or borer of the sea. It is true it appears only rarely, but in such numbers and is so destructive that commissions of learned specialists have been appointed to study his life, history, and devise means for protecting timber from his attack.

About the middle of the last century it was discovered that a shell-fish was industriously perforating submerged piles and wharf timbers. A hasty examination showed that at many places the very bulwarks of Holland's safety were literally honeycombed. The discovery of this condition threw Holland into dismay—its continuance meant destruction, while the ignorance of any preventative stimulated the fear that the worst possible calamity was near at hand.

Fortunately the very means which were taken to protect the piles unwittingly assisted in the extermination of the terrible pest. Large

headed nails were driven in the wood so close together that they practically gave it a coat of mail. But chemistry was more potent than physics, for the oxides from the rusting metal was disagreeable to the teredo and his detention near the surface of the wood so exposed him, that a few severe winters finally destroyed them all.

However, they have reappeared and caution has kept the more important piles covered in part by copper sheeting along the dykes of Friesland, enough copper has been used for this purpose to cover the entire dyke.

A worm had made Holland tremble—a triumph denied to the tempests of the ocean and the anger of Philip of Spain.

In this fragmentary country, broken into parts by lakes and cut into pieces by rivers and canals, interests centered around localized systems of hydraulics. Thus one community was a unit in these vital matters of sustenance and self-preservation, and its people naturally felt a greater allegiance to the local government than to a centralized power. Then from the liberty of the canton or village a single differentiation lead to the liberty of the individual under such conditions, an empire could never have come into existence, with such an origin the United Netherlands are indissoluble.

One never combats nature with abstractions. In Holland, man is inevitably kept face to face with realities by the watchful care which his very existence demands and the material obstacles which must be conquered at every step. Patriotism never becomes dormant because the face of the land shows in its scars its own history, and love for home glows at the reckoning of the cost of its retention.

One saw this little nation, almost imperceptible during the 16th century on the map of the world, build dykes, and contest with the sea for supremacy. In their struggle against Spain they preferred to treat with the sea than with the Duke of Alva ; and when no longer able to cope with superior forces, they cut the dykes and flooded provinces, preferring to drown themselves with the land of their creation than live upon soil outraged by the feet of foreign foes.

Holland, always protesting against certain physical laws which threatened its existence, seeing by example the danger to man's temporal estate by the observance of the doctrine of "laissez faire," and realizing its need to place human wish in apposition to the hallowed forces of nature, was soon on the highway to religious revolt. Therefore, when the reformation appeared, protestantism answering to the active instincts of the Batavian race, became an accepted creed.

If the life of the nation has been influenced by the structure of its

land, still more has the individual yielded to his environment. In most countries wealth begets idleness. In Holland, never. Usually when nature is prodigal with her gifts, when crops readily grow while man reposes, he becomes selfish. Not so in Holland. A little crevice in the dyke, unnoticed for a few hours, might permit the devastation of a distant city, and even with the most watchful care, the possessions of one day are no guarantee of the wealth of the next.

Without the Dutch, there would be no Holland. This country is in truth and in fact their creation and they have the undeniable right to look upon their work and say "it is good."

Without science and industry, such a land would never have beheld the the light of day, and but for the incessant vigilance of its people it would soon perish. Its creation is a miracle of human genius; its preservation is a monument to his skill.

We have discussed the conditions under which this annex to the continent was made; the technical processes employed by the people to keep back invading waters; how they have built cities upon moving sands which the sea claimed and still demands; the marking of the river courses and how they were constrained to keep within the same: how agriculture was introduced into ancient lake beds; and in short how the Netherlands, with originally nothing more than a vacillating admixture of land and water, has become the world's model pasture. The genius of Babylon proudly boasted, "it is I who made the Euphrates," but culture upon its banks did not continue long after its making. The genius of Holland even in a greater sense can say, "it is I who made the Rhine, the Meuse, and the land through which they flow," and culture upon their shores grows with the passing years.

This constant struggle places success as the only accepted goal of every effort, but the effort, to be availing, must be concentrated. One man working alone cannot build a dyke, neither can he check in time a threatened break. Labor, therefore, can never be selfish and individual. The lesson learned in the war with the sea becomes a guide in organizing the battle with competition, and guilds and corporations are the result.

When the surging waters approach dangerously near the vulnerable points of an important dyke, every shovelful of earth must count for something, the opposing forces must be placed and used to the best advantage and safety is assured only when obedience is obtained. Discipline, therefore, is a shining Dutch trait.

While one community is rejoicing over its escape from inundation, the people near by may be counting up their losses in life and property;

thus the one sympathized with the other, when the latter was felled by the bolt which missed the former. This possibility of a coming misfortune makes every one generous, and the hundreds of charitable institutions in Holland prove that this generosity assumes tangible form.

We, therefore, find a nation, from its war with the sea made up of citizens who are rich without being insolent, learned and accomplished without being proud, earnest in religious matters, but not bigoted.

THE NORTH RIVER BRIDGE.

G. LINDENTHAL, C.E.

Chief Engineer of Proposed North River Bridge.

GENTLEMEN:

The invitation to speak before you to-day on the subject of the North River Bridge took me quite unawares, as I had but just returned from abroad, and so have had no time to prepare a lecture. But such information on the subject as I can give you on short notice will, I hope, not be without interest to you. In what I shall have to say, I will confine myself to the general technical features of the undertaking, assuming that you are sufficiently acquainted with engineering subjects to follow an exposition of this character.

We will assume that the location and length of span for the bridge have been determined, and that the problem before us is the designing of a suitable structure for the requirements presented by the local conditions.

At the place where the bridge will be built, the river is less than 3000′ wide, and will be crossed by a single span 3100′ long from centre to centre of towers. The end or land spans will be each 1850′ long, because of the necessity of overhead crossings with intersecting streets, and because good foundations for the anchorages could not well be had nearer to the river shores.

A single span over the entire river does not involve a wasteful construction, as had been claimed by some engineers, who had not carefully studied the situation. On the contrary, it is for the proposed location the cheapest plan. The river bed consists of mud of great depth, sand and then rock, which near the middle of the river is from 260 to 300 feet deep. The pier foundation for such an important work would necessarily have to rest on a rock, and in this case would cost more than the extra cost of a single span over the entire river. From my investigations, I had come to the conclusion more than ten years ago, that the concession of a single span over the entire river to the commercial interests of the Port of New York, which were much opposed

to a pier in the river, could readily be made without involving extra cost. But my opinion was not generally shared, and doubt was even expressed as to the practicability of a span of 3000 feet at any cost. The controversy has, however, been settled authoritatively, after full investigations at the instance of the United States Government.

The present traffic requirements demand eight railroad tracks, and the bridge will be so designed that the number of tracks may be increased in the future to fourteen.

The design before you is the one which has been adopted by the North River Bridge Company. It is for a stiffened suspension bridge, of fourteen tracks, eight of which will be placed on the bridge as soon as erected, the remaining six to be laid in the future as the traffic may demand.

Changes of detail have been made from time to time since the plan was first published. Every progress in methods of construction or in steel manufactures, which can hasten the work of construction, when once begun, is availed of for improving the details, some of which could not have been manufactured at all only a few years ago.

At the time when this design was first proposed it was thought somewhat bold. It is true that we have a railway suspension bridge over the Niagara gorge, which has carried the traffic successfully for nearly forty years ; but on account of failures in suspension bridges elsewhere, the opinion was generally held that suspension bridges are unfit for railroad purposes.

As a matter of fact, a suspension bridge of long span is a most suitable type for railroad purposes. A suspension bridge is an inverted arch bridge in which the arch is in tension and in stable equilibrium. An erect arch, on the other hand, would collapse unless rigidly braced laterally and vertically. The centre line of gravity in an erect arch bridge is above the points of support, and therefore it is in unstable equilibrium. In the suspension bridge, the cables keep their form without any lateral bracing and the bridge is not in danger of collapse, but subject only to vertical deflections.

In designing a suspension bridge for railroad purposes the first thing required is to fix beforehand the allowable amount of vertical deflection from live load. Any degree of stiffness may be given to a suspension bridge without impairing its safety. It may be made as rigid as a truss bridge or as limber as a wagon chain. Between the rigidity of a truss bridge and the flexibility of a suspension bridge without any stiffening system, every degree of stiffness is possible. It is the engineer's business to determine the amount of stiffness which the bridge must have,

suitable to the loads over it. Having fixed the allowable deflection from live load, the stiffening system can then be computed and designed accordingly. The greatest deflections, plus and minus, will be at the quarter points of the middle span, and at the half points of the end spans.

The amount of deflection caused by the live load will depend on the manner of loading assumed. In the present instance of eight tracks, the worst possible condition of loading would occur for eight heavy trains passing abreast over the bridge in one direction. Although this condition of loading is not likely ever to occur, yet it must be considered as a possibility for the dimensioning of the bridge. The allowable deflection in the quarters was assumed to be 29 inches for this bridge under the most extreme conditions. Ordinarily the deflection will be only a small fraction of it, rarely more than three inches.

The live load for the North River Bridge is taken at 3000 pounds per linear foot of track. This is the same load usually taken for railroad spans over 350 feet. The office of resisting deformation caused by such load is, in this design, entirely thrown upon the suspended arches. Each arch consists of two cables placed vertically 65 feet apart and braced together. Two parallel cord trusses are also used, but not intended to act as stiffening trusses.

Where such extreme conditions of loading are assumed it is admissible to use very high unit strains, equal for the steel wire to one-third of its ultimate strength.

The diagonal bracing between the cables is not strained by a uniform load. The stresses in it will be due only to moving loads and to temperature changes.

For the dead load the diagonals between the cables are not necessary and might be left out altogether. Under the assumed extreme conditions of live load, very high unit stresses, up to the elastic limit, may be used for them, without the least danger to the safety of the bridge. If any of the diagonals should break, nothing worse could happen than the sagging a few inches of the cables under the moving load, but this is not probable.

An important question in this design was, how the suspended arches for the middle span should be arranged: whether as arches with fixed ends, for which the cables would be continuous over the towers; or as arches with hinged ends; or as three hinged arches, (one hinge being then in centre of the middle span). The question was thoroughly investigated. It was thought at first, that as the temperature strains would be very considerable, special provision would have to be made to

eliminate them, by using hinges on the towers and in the centre of the span. My investigations satisfied me that, although by using three hinges the temperature strains might be eliminated from the ends and the centre of the arch, it was not true, contrary to the received theory on the subject, of the rest of the arch, and that the temperature strains, particularly in the quarters, were in fact greater than if the centre hinge did not exist; there would be furthermore great secondary strains caused by the friction of the hinge movement. For these reasons I decided *not to use* the form with three hinges.

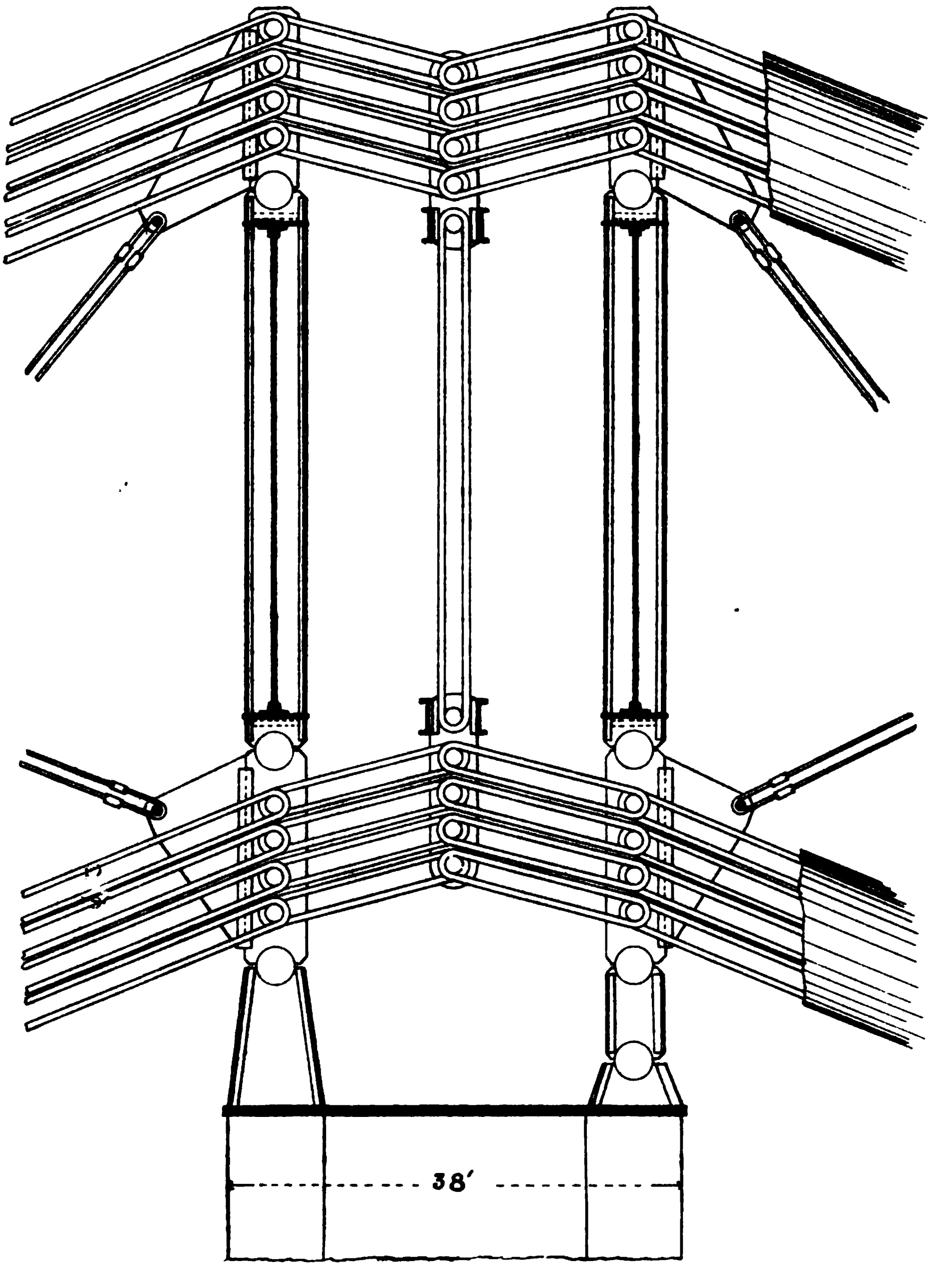
Again, in the arch without hinges, the large bending moments near the towers would have required there a large increase in the section of the wire cables; and it also involved the danger sometimes of compression strains in them. To avoid the possibility of such occurrence, I decided to use a special form of hinge, since the ordinary form is not practicable for wire cables of the huge size required for this bridge. The form is shown in Plate I.

It will be readily understood that for a bridge of this very long span, the cables must be made of steel wire. They cannot be made of bar iron or steel eyebars, having a strength only of from 50,000 to 65,000 pounds per square inch. With steel eyebars the weight of the superstructure would be over two and one-half times heavier than for cables of steel wire. It would mean that the steel towers would have to be made two and one-half times heavier, the foundation two and one-half times larger, the anchorage two and one-half times larger, and everything else in proportion, increasing the cost to a prohibitive limit. But by using steel wire of a high strength, namely, 180,000 pounds per square inch, the dead weight is very much lessened. Such wire has just three times the strength of ordinary bar steel, as used for structural work.

In the Brooklyn bridge the wire used was made of hard crucible steel ⅛ inch in diameter, having a strength of 170,000 pounds per square inch. Steel wire with a strength of 320,000 pounds can be obtained, but such wire is drawn down very fine and is expensive. In order to reduce the surface of rusting, and to lessen the number of wires for each cable, I had from the start decided to use No. 3 Birmingham Gauge, which has a diameter of ¼ inch, and hence is twice as thick as that in the Brooklyn bridge.

When I first proposed Birmingham No. 3 wire, with a strength of 180,000 pounds, none of the wire makers thought that they could make it, except at a high price; but in the last few years great progress has been made in the drawing of wire, and I have the assurance of wire

TOGGLE CONNECTION. Plate I.



manufacturers that it can now be had at a comparatively low cost. The wire will be made of hard open-hearth steel. It is so stiff that a piece 20 feet long held up by one end deflects but slightly.

In each cable of the Brooklyn bridge there are about 6,300 wires. Their accurate adjustment required much time. It took two years to spin the four cables. In the North River bridge another method of cable construction will be used, to save as much time as possible in their manufacture and erection. Each cable will consist of four chains above each other, and coupled by steel coupling-plates at each panel point. The pin-connections will be similar to that of an eyebar chain, with only this difference, that in this case the eyebars will be made of wire. The wire links will be of different size, running from 200 to 400 turns of wire around cast steel shoes, one at each end. In the process of manufacturing them at the shop, it will be possible to regulate the length of the wires with great accuracy, and the cast steel shoes will be bored out to exactly fit the hollow steel pins. The finished wire links will be wrapped every 5 to 10 feet with soft wire, which will keep them in shape for transportation and erection. As soon as the towers are finished, the cables can be erected rapidly, independent in any way of the weather.

The vertical distance between the two cables is everywhere the same, so that the vertical braces will all be of the same length. The pins at each panel point of the four chains are also in a vertical line, and connected by coupling plates. In chains arranged in this way no local bending strains can occur from any deformation of the cables during erection, or later due to moving loads.

The cables will be strong enough to carry eight tracks at first, and can afterward be increased in strength to carry fourteen tracks.

By referring to Plates I and II, the arrangement of the wire cables will be seen. For the eight track bridge, the four chains of each cable will first be all of the same size, equal in width to the upper and lower chain. The pins of the two centre chains will be longer than and project beyond those of the upper and lower chains. Later, when more tracks will be added on the bridge, the cable section can be increased by placing more links on the projecting pins of the two centre chains.

The wire cables will be surrounded by a water tight steel casing, to protect the wire against the effects of the weather, and so arranged that it can be taken off in sections. The outside diameter of the casing will be 13 feet. A man will be able to crawl through all parts of the cable for inspection. The steel casing will also protect the wires against uneven heating from the sun. It cannot be denied that wire cables of

the form used in the Brooklyn bridge are subject to internal stresses by reason of the uneven heating from the sun. It was observed during the erection of the Brooklyn bridge, that before the floor construction was suspended from the cables, these would show a torsional motion in the sun of about 30 degrees from morning to evening. The sun would heat up one side of the cable, while the shady side would remain cool. The warm wires would expand, and, of course, drop into a lower position of equilibrium, while the cold wires would rise, causing the rotation mentioned above. After the erection of the superstructure, the cables could not turn, and thus they are subject to internal stresses.

A further point to be noted is, that in the Brooklyn bridge the wires are zinc-coated. It is almost impossible to prevent abrasion of the zinc during erection. The cables, notwithstanding the most careful painting, will take up moisture from the air, partly by condensation and partly by capillarity, and if there should be a spot unprotected by zinc or paint, the metal at that point is liable to rust from galvanic action much faster than without the zinc coating. Hence I propose to have the wires of the North River bridge not galvanized, but thoroughly oiled, and accessible to inspection at all times.

The diagonal bracing between the upper and lower cables will consist of adjustable tension members, composed each of a number of smaller sections, as shown on Plate I. The heaviest diagonals will thus consist of 18 bars, 2¾ inches square, which is yet a convenient size for sleeve nut adjustment, and may be easily manufactured with present plant.

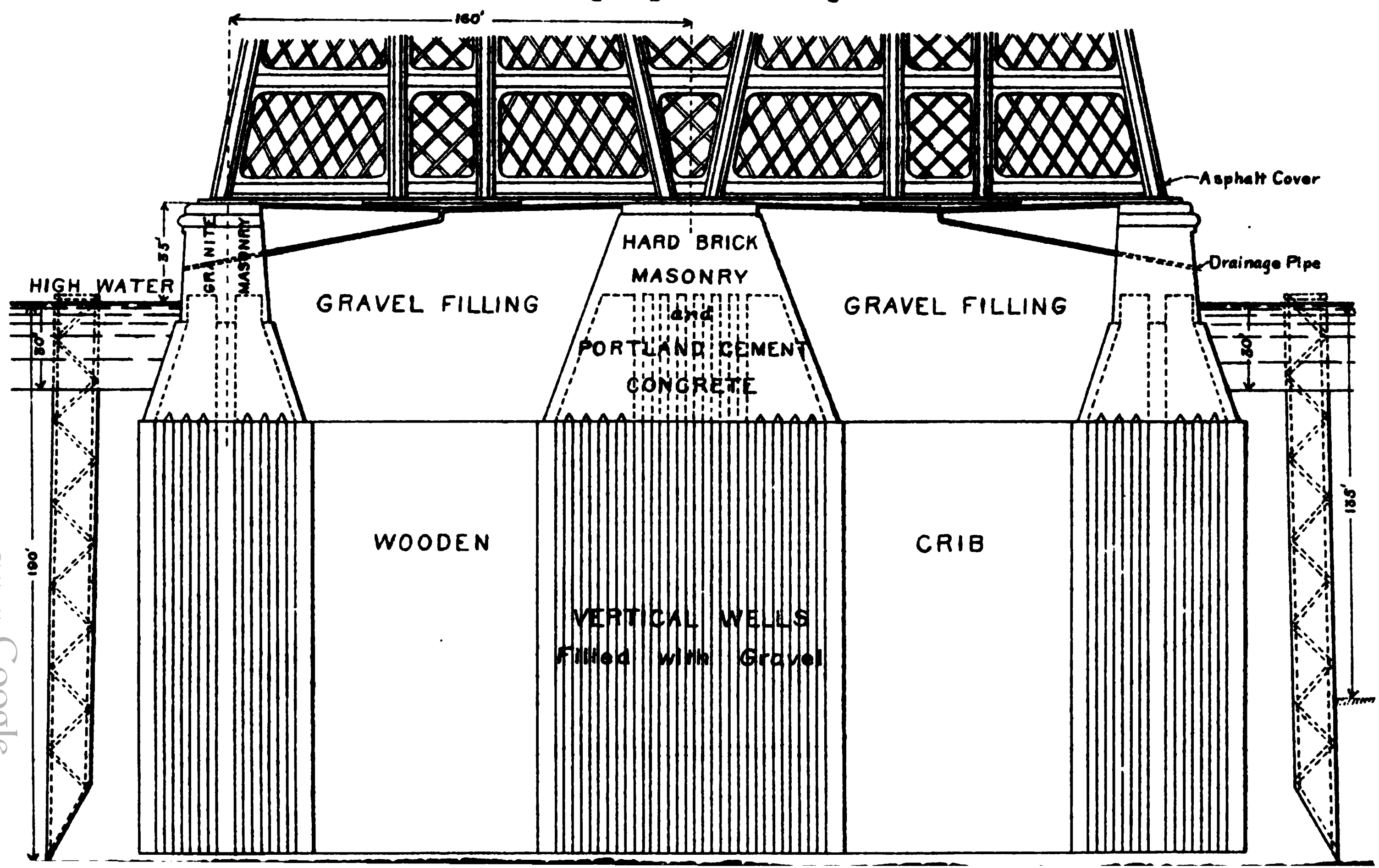
The method of suspending the track platform from the cables is shown in Plate II. The cross section is rectangular, and braced with an arch at every panel point. See Plate II.

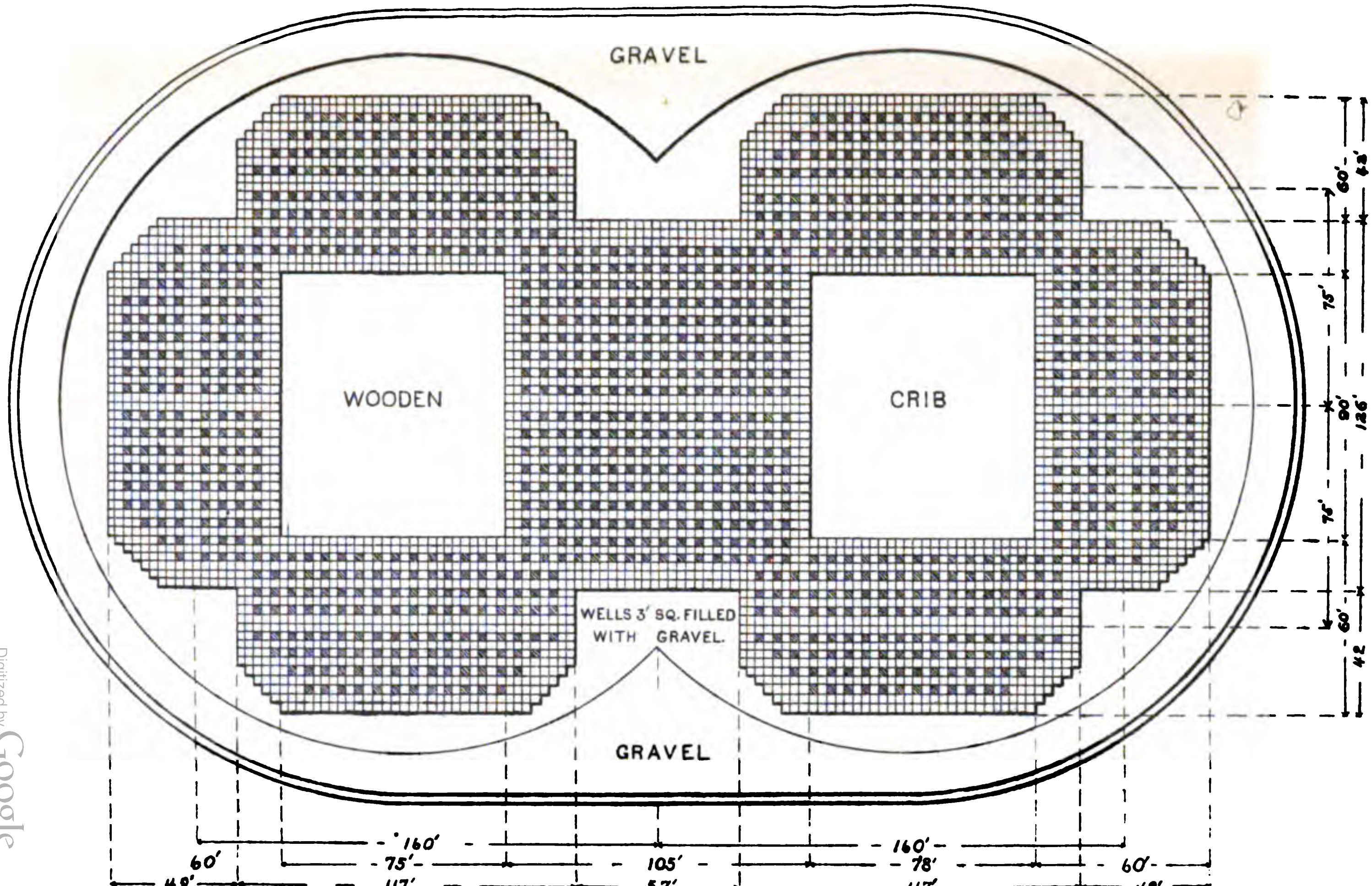
It is necessary to make the thickness (or height) of the floor construction as small as possible. It will be in this case 6 feet. The girders or stringers carrying the tracks (proportioned for the heaviest locomotives on the basis of the usual low unit strain) will be placed directly under the rails, and will be carried by the cross girders, or floor beams, suspended at three points from the arch. The arch will therefore serve the double purpose of carrying the floor beams and of bracing the upper to the lower wind truss. The upper wind truss is under the promenade and the lower wind truss under the lower floor beams.

The upper and lower wind truss will thus deflect sideways, necessarily to the same amount, and therefore the chord sections of the upper and lower wind trusses will be of equal size.

The wind trusses extending from tower to tower will be continuous and of uniform chord section. The assumed wind pressure is one ton

CROSS SECTION OF FOUNDATION.
At Right Angles to Axis of Bridge.





PLAN OF COFFER DAM AND WOODEN CRIB.
Coffer Dam of Iron, braced with Wood and filled with Gravel.

per lineal foot of bridge. The chords of the wind trusses are further utilized by means of vertical bracing for stiffening trusses, but only to the extent of distributing the concentrated load of locomotives over a greater length, and so that the cables receive it as a uniform live load similar to that assumed for the train loads.

Although the bridge might provide also for wagon traffic, it is not intended to do so, because wagons could not get up to the great height of the bridge from the streets without elevators, which would be less convenient than the present ferries.

Plate I shows a view of the bridge as it will appear when completed. The towers are 580 feet high. The piers are 360 feet long and 180 feet wide.

FOUNDATIONS.

Having stated to you some of the characteristic features of the superstructure, as it will appear, I will add a general description of the proposed foundations for the two great towers:

The foundation for the towers on the New York side will be 190 feet, and on the New Jersey side, 120 feet below water, as shown by borings. The water is here from 20 to 25 feet deep. The river bottom consists of soft mud, then clay mixed with sand, some gravel and then rock, to which the foundation must be sunk. It is too deep for the pneumatic process. The air pressures in the working chambers of the caissons would be from 50 to 76 pounds per square inch, whereas the greatest pressure endured so far did not exceed 49 pounds per square inch, resulting in a large percentage of mortality among the workingmen.

The method to be followed for these deep foundations will be as follows: A coffer dam, constructed of iron and timber of the form shown on drawing, Plate III, will be sunk, by forcing water at a pressure of 80 to 100 pounds per square inch through 2" tubes placed 10 feet apart around the exterior edge (or cutting edge) of the dam. By this means the material in the bottom will be forced to the interior of the dam, from whence it will be removed by sand pumps and dredging. As the dam is sinking, the exterior and interior iron lining is riveted up, the bracing put in, and the space filled with material dredged out from the foundation.

Before sinking the coffer dam, the surface of the rock on which it is to rest, will be ascertained by means of borings, and the bottom or cutting edge of the coffer dam given such a shape that when it reaches the rock it will bear on it all around. The coffer dam is merely for the

purpose of keeping the mud out, and is not intended to carry any part of the weight of the tower.

The rock near both shores has an inclination of about 10% towards the middle of the river. After all the mud has been taken out, the rock surface in the interior of the coffer dam will be levelled up with concrete sunk below water, and broken stone, to a perfectly level surface. As the area is very large, nearly two (2) acres, the soundings and levelling will have to be done carefully and systematically to insure an even surface on which to rest the timber crib, which will be built up inside the coffer dam. Two-thirds of the section of the crib will be solid timber and one-third will be open wells three feet square to be afterwards filled in with gravel and concrete. This proportion of wood and stone will give a weight equal to the weight of water displaced, and hence the crib can be built up with vertical sides. If the crib material or submarine foundation were heavier than water, its sides would, for reasons of economy, be inclined, because the base should be larger than the top for equal pressures per square foot. The coffer dam would have to be larger for the larger base, and the whole foundation would be much increased in cost.

The timber for the crib will be 12"x12", accurately jointed, and planed to exact size and connected by drift bolts. When finished the floating timber crib will stick out of the water about one-third of its depth, namely, about 50 feet for the New York side and 30 feet for the New Jersey side. On top of the timber crib will be placed masonry of hard burned brick, or clinkers laid in Portland cement, containing wells corresponding with those in the timber crib, which will be afterwards filled with concrete. The whole weight of the foundation (crib and masonry), with the open wells in it, will be so proportioned as to keep afloat, so that all work on it can be done above the water level and without resort to pumping. The crib will gradually settle down on the level bottom prepared for it and the masonry, which will be of granite above water, finished to coping about 35 feet above water.

An important matter to consider with this kind of foundation will be its compressibility. No experiments have ever been made to determine the ratio to be allowed for compression in timber cribs, and, therefore, special experiments on a large sale will have to be made to determine the compressibility of this kind of crib foundation. Then we will know how much higher the masonry will have to be built above water, to allow for final settlement of foundation.

As the diagonal bracing between the upper and lower cables will be adjustable, any settlement of the tower foundations after completion of

the bridge will be harmless. It will not permanently affect the strains in the cables or towers, or in other parts of the steel structure.

The foundation here described has the advantage that it can be built quickly. Every part of it is constructed in the open air, and can be readily inspected. The foundation of the New Jersey side will be shallower by about 70′, but otherwise of the same character.

The shape of the coffer dam is shown on plan, and it is designed to safely resist collapse and deformation from the mud pressure. The space between the crib and the coffer dam will be filled with gravel. The wooden part of the crib will stop below the mud level, or 40 feet below water, and, therefore, will be safe against the attacks of the teredo.

PECULIARITIES OF END SPANS.

Each end span is 1,850′ long, which is larger than the center span (1,600 feet) of the Brooklyn bridge. The bending moments from live load in these end spans may become larger than any that may occur in the middle span. The cable sections and the bracing of the end spans would need to be heavier than in the middle span, and this is not desirable. To get over this difficulty, an intermediate support is provided. By referring to Plate I, you will notice columns placed in the center of the end spans. The columns carry no part of the dead load, and their only office is to assist in carrying heavy concentrated live loads, for instance, heavy coal trains, which would otherwise cause the end spans to sag. The columns are designed to take either tension or compression. When the middle span only is loaded the end arches will tend to rise and the columns will act as anchors and be in tension. With this arrangement the horizontal component of the stresses in the cables from dead and live load can be kept within the same limits in the end span as in the middle span. As to the special methods of computation for this arrangement, I may have an opportunity at a later time for their explanation and discussion.

ERECTION.

I will now briefly explain the method of erection for the superstructure. The towers will be erected first, and it is obvious that no false-works will be required for their erection. Temporary wooden towers 80′ high will be placed on top of the steel towers, and wire ropes will be stretched over them between the anchorages. In this way a temporary bridge will be formed, from which the first set of wire links may

be erected, by means of temporary suspenders, which will support the cable pins in proper position. One set of links, of the topmost chain, alternating one and two links, is in this way connected up from anchorage to anchorage, and thus will be self-supporting, without further assistance from the temporary wire rope cables. More links are added, slipping them sideways on to the pins. Chain after chain in both cables from the topmost to the bottommost will thus be gradually erected. No adjustment is required and the work can proceed rapidly. The erection can be carried forward day and night, and 160 erecting parties can gradually be placed on the cables at the same time. The four cables can easily be erected in this way in four months, as compared with the twenty-four months required for the four cables of the Brooklyn Bridge.

COMPARISON WITH OTHER LARGE BRIDGES.

The anchorages with the buildings on top will be over 250′ high and the towers 580′, or more than twice the height of the Brooklyn Bridge towers. The enormous and yet harmonious proportions of the North River bridge will be apparent from the plan. The relative economy of designs may be observed by comparing the respective weights of steel per lineal foot of track, for the different bridges given in the following table:

TABLE—SHOWING WEIGHT OF STEEL PER UNIT OF SPAN.

Steel Bridges.	*Length of Span.*	*Steel per lineal foot of bridge.*	*Steel per lin. ft. of track.*	*Steel wire per lin. ft. of bridge.*	*Steel wire per lin. ft. of track.*
North River Bridge, for eight tracks.	3100′	36000 lbs. including towers and anchorages.	4500	12800	1600
Brooklyn Bridge, equivalent to two tracks.	1600′	6200 lbs.	3100	2200	1100
Forth Bridge, for two tracks.	1700′	18400 lbs. including towers.	9200	. . .	. . .
Po'keepsie Bridge, for two tracks.	530′	7300 lbs. excluding towers.	3650	. . .	. . .
St. Louis Arch B'dge, for two tracks.	520′	7000 lbs.	3500	. . .	. . .

You will notice that the weight of steel, including the towers and anchorages, per lineal foot of track (4500 pounds) for the North River Bridge does not exceed that of merely the superstructure of a 450 feet span. It means that, at the least, no more steel will be required for the superstructure of this bridge with a span of nearly one kilometer, than would be required for a series of truss spans 450′ each, whose sum would equal the length of this bridge. This remarkable economy is due to several peculiarities in this design. For instance, to the entirely proper use of high unit stresses in members, very rarely, if ever, receiving the calculated maximum strains. Further, the wind trusses need not be much heavier for the eight track (or fourteen track ultimately) than for a double track bridge. The greatest economy, however, will result from the use of hard steel wire for the cables. As mentioned before, a span of 3100 feet would not be practicable without steel wire, of which, as per table, 1600 pounds per lineal foot of track will be required in the North River Bridge, as against 1100 pounds in the Brooklyn Bridge; although theoretically it would have been supposed, that the North River span, being nearly twice as long, would require about four times the weight of steel wire, (4400 pounds per lineal foot of track). Only from actual design, however, can the weight be correctly obtained, as was done in this case.

As regards the question of rigidity (meaning thereby resistance to vibration), I will merely mention that a really stiffened suspension bridge has the smallest rate of vibration of all metallic bridge types. Rigidity is no indication of safety, it is merely a desirable quality in a bridge for comfort. A stone arch bridge may be unsafe and near collapse, but it will be rigid and more comfortable for walking and riding over, than an unstiffened and limber chain bridge, which otherwise is absolutely safe. The stable equilibrium of a suspension bridge, assisted by a rational system of stiffening, resists vibrations better than the special bracing against vibration required in other bridge systems. As an illustration, take the insufficiently stiffened, single track railroad suspension bridge over the Niagara River (now nearly forty years in use) and the double track cantilever bridge, located a few hundred feet above it. Although the former deflects more under trains than the latter, it is freer from vibration than the cantilever bridge. Compare the two largest existing bridges in the world, the Brooklyn Suspension Bridge and the Forth Cantilever Bridge. Although the Brooklyn Bridge superstructure weighs only one-third of that of the Forth Bridge, and although no special bracing to prevent vibrations was used in it, as against the very careful and thorough bracing in that regard of the

Forth Bridge, the vibration from passing loads in the former is no greater than in the latter.

The North River suspension bridge will be free from vibration, not only by reason of the rational method of stiffening between the arches, but also by reason of its great weight, which will be about double that of the Forth Bridge, and nearly six times that of the Brooklyn Bridge.

Moreover, the combination of the suspended arch with the auxiliary parallel chord girder will greatly add to rigidity, because the two systems have different periods of vibration, interfering with and destructive of each other in that respect. In other words, the rigidity of the North River Bridge will be little different from that of solid ground.

Its cost, estimated for eight tracks, at present prices for labor and material, but without right of way, interest and administration account, will be $21,000,000, or at the rate of $360 per lineal foot of track, as against $1,100 in the Forth Bridge under the same limitations.

In closing my remarks, please observe that in an hour's talk I could give you only an outline of the plans for the structure, and a few indications of the practical considerations, which must guide the engineer in designing it.

There is no doubt that with an improvement in the financial condition of the country, which has been unfavorable for the last few years for any new undertaking that requires very large amounts of private capital as this one does, the great work will be progressed to completion, and then the time will be fitting to enlarge on the details of its construction, which although extensively studied and prepared in advance, may be further improved through better methods of fabrication.

The engineer who loves his profession will never cease to be a student, with the same eagerness to learn, to improve, and to advance, as when he was a beginner like yourselves.

THE CONSTRUCTION OF SKY-SCRAPERS.

C. T. PURDY, C. E.

The day is coming, it seems to me, when the designing of bridges and the designing of buildings will be kept as entirely separate and distinct as Hydraulic Engineering and Structural Iron work are now,—when men, who devote themselves to one subject, will do little or nothing with the other, and this in spite of the fact that both concern Structural Steel and the same engineering principles. It seems to me so because both bridge construction and house building involve their own peculiar interests and considerations, some of them quite outweighing the points they have in common. The business men concerned in either line of construction are not likely to be interested in the other, and it is not easy to make friends and gain clients in more than one field of activity at the same time. To begin with, then, if you are to be a designer of Structural Iron work you will probably not work in both fields. It will be bridges or houses, one or the other, and not both.

The Engineer who gains a place of responsibility and importance in house building work will in all likelihood have to deal with three classes of problems; problems in stresses, problems in business, and problems in architecture.

The columns in a building can be arranged in several ways, and it is necessary to decide which way is best and which way is most economical. There are several kinds of columns, and some one must decide which is the most desirable. The beams also can be arranged in several ways, and one way is pretty sure to be the best. To decide these points intelligently and quickly requires a knowledge of stresses in beams and girders, and in columns; not an indefinite, uncertain one, but a *perfect* knowledge of all the principles, formula, and facts concerning stresses of this kind. These things should be so definite and clear in the Engineer's mind that he can work with them rapidly, without stopping to think or remember.

He should also have a perfect and handy knowledge of how the work

should be detailed, and how it can be manufactured. I remember once telling a client that a certain amount of space would be required for a piece of riveted work and being very courteously informed that I must be mistaken, for a similar thing had been put in another building in less space. I found an opportunity a few days later to examine the design and drawings of the work referred to, and discovered that the rivets are actually strained up to 18,000 pounds to 20,000 pounds per square inch of section. Examples might be multiplied indefinitely, showing how important this knowledge of detail is. When the designer first lays out his plans in the rough, he must be able to look clear through to the final completion of his work, with sufficient comprehension of what will be required in every detail to do practical work. It is really astonishing how many impracticable, and sometimes impossible, things are put on to architects' general drawings, where inexperienced men are employed. A last year's graduate told me a few days ago that he had been looking over a drawing which he made for his thesis, and that he had discovered a number of details in it which made the proper erection of the work impossible. I will show you later a few pictures of portal bracing. The perfection of the details of the arches was an evolution. One of the earlier designs required that the work should be erected when the columns were put in place. We got the rest of the iron work, but could not get the arches, and the entire construction was stopped. The same arches were put into another large building, designed by an inexperienced man,—inexperienced, at least, so far as the arches were concerned—and when they were first loaded on the cars at the manufacturer's they were in such large pieces that the railway company refused to haul them.

The Engineer should also know the prices of all kinds of steel and iron, the relative cost of different kinds of shop work, and the values of other building materials. We have now three kinds of walls; the wall that carries its own weight and is simply anchored to the interior construction, the wall that carries the adjoining floor in addition to its own weight, and the wall that is carried from floor to floor on a steel frame and does not even carry its own weight. The Engineer must know the cost of the masonry to properly judge between them, as he may quite possibly have to do. A half dozen different materials may enter into the construction of a floor,—a change in the iron may cause a change in some of the other materials,—and the economy of the construction concerns the value of them all.

He must also know the weights of all kinds of materials used in a building, and how they are put together. The accurate estimate of the

dead load of a building and its proper distribution, is in many places of exceedingly great importance. If the building is a skeleton construction, and terra cotta is used to any extent in the design, he can do no acceptable work without being quite familiar with the manufacture and use of the terra cotta. And there are other matters larger and more important than these. He may have large trusses to design, and lateral bracing, and complex and difficult girder work, involving at times not only very difficult problems in stresses, but all his ingenuity and inventive genius to so arrange such work that its construction shall not injure the building for its use or as an investment. Great buildings as a rule are not erected as we would build a house to live in, with only a thought for its convenience and comfort, but they are planned and constructed to earn a fixed and definite income, and the success of both the architect and the Engineer depends very largely on their ability in this direction.

The college training will be necessary in the future more than in the past, in preparation for successful practice, and this is especially true in the line of structural designing. The man that is not college bred soon reaches the limit of his professional capacity, and beyond that he is heavily handicapped. I say this because I want to say so much about the importance of the practical training that generally comes after the college work, that I fear I may seem to undervalue the theoretical education, which I am sure I do not. In the particular work of which we are talking now, this practical training requires more time than is required for efficient service in most other lines of engineering. There should be draughting-room practice, experience in mill and shop, and service in erection, not of structural steel work only, but of every part of a building. Besides this, the student should gain a constructional knowledge of architecture before he can be confident of his own judgment and ability. The actual labor of making drawings is purely mechanical, it is not what you are aiming at, and yet it is the first thing you may have to do.

Your college training should teach you to respect your ignorance, and show you how and where you may inform yourself on any particular question. It will give you some definite knowledge, and most of all, a more capable mind. When the days have gone by, and the larger responsibilities come, you are going to realize and feel its value, but it will not help you much in the draughting room, or in the shop, or in erection. At best it can but introduce you to these things. You can be taught how they ought to be done, and even try your hand at them, but neatness, skill, accuracy and the rapid use of hand and head cannot

come without days and weeks, and possibly months of the actual doing of the things. When you come to the larger problems you want not only that invaluable mental training you are now getting, but a quick, almost intuitive judgment that should be the product of these additional years of your practical preparation and education. I don't want to discourage you by putting the days of achievement further away, but I want to inspire you to a greater success finally. The more you give in preparation, in time, in effort, in hard work, in completely covering all the ground, the more splendid will be your success. The world is full of half prepared men. If I were one of you, yet with my actual knowledge of how things go, I would work five or six years in these lines before attempting at all to practice alone, or to take anypermanent position.

A knowledge of shop practice may be gained through employment as an inspector, and of erection, through employment as inspector or superintendent. A knowledge of how things are actually made ought to extend to fire-proofing, terra cotta, brick and cut stone work, as well as both structural and ornamental iron work. I remember I had to have several beams pulled out of a building before I learned how to make an opening for a stairway properly. The best way to familiarize oneself with an architect's drawings is to work for a time in some good architect's office. If the opportunity of actual service in any of these lines is wanting, observation, reading, and study should be made to compensate as much as possible for it.

I do not forget that complete success requires business ability, hard work, patience and other things, nor that a man can be successful in business without being a good engineer. I want to be understood—I have been speaking for professional perfection.

The construction of a large building might be divided into the following divisions: Foundations; structural iron; masonry; fireproofing; plumbing, sewerage, and gas; heating and ventilating; electric work; elevators; power plant; carpenter work; plastering; painting, glazing, and tinting; sheet metal and skylight work; hardware; ornamental iron work; marble and mosaic; fireplaces and mantels.

The first two,—foundations and structural iron, stand for strength. If they can be made to give a building stability and permanence, without interfering with the design, they should be entirely subordinate to it; but if not, the design should be modified, and the strength of the building should be made of first importance. Their consideration belongs primarily to the realm of structural engineering. The same is also true of masonry, so far as it is used to add strength and stiffness

to the building. The reveals, projections, and ornamentations of the walls are for appearance,—the art of architecture.

The plumbing, sewerage and gas, heating and ventilating, electric lights, bells, elevators, and power plant are for the service of the building, and their proper design and construction is best attained by the employment of engineering experts. Each of itself is a specialty, and we have now some very able men gaining good incomes in consulting work along these lines. Occasionally a man can be found trying to be an expert in all directions, but I am not intimately acquainted with anyone who has yet succeeded in that way.

All the other divisions should be specified for in any architect's office, without recourse to outside help. This employment of consulting experts was at first considered disgraceful, but architects are changing their minds, and the day is coming when the employment of experts will be the rule instead of the exception. It relieves the architects of a large burden of detail that counts for little with the world at large, and in the time saved he can give more attention to the artistic side of his labors, and really do more business with the same amount of machinery.

The structural Engineer must understand and anticipate the wants of the plumber and steamfitter, or he will surely find that he has interfered with their work. Elevator construction often incurs heavy loads in the middle of a building, which must be cared for, and the framing about elevator hatchways and stair wells must be based to some extent on the way the enclosures and facia and ornamental iron is to be constructed. The marble finish has often to be cared for, and sometimes fireproofing and partitions must have special support. And, indeed, there is little or nothing in the entire construction of a building that may not occasionally demand some provision or some concession in the structural iron.

The last decade has seen a great change. Steel made greater things possible, and with greater things came greater responsibilities, new methods, and a new architecture. Architects, mechanics and builders have had to adopt new methods. The questions that are asked these days are not about the experience of these men as a whole, but about their experience in the new types of construction. I am taking it for granted that you understand the force and extent of this evolution in house building. In the old type of construction, if columns were used at all, or girders, they were on the interior and entirely supplemental to the masonry in their office of giving strength to the building. In the perfect type of the new building this is reversed. A great frame

work of steel, strong in its own strength, is first erected; then the floor arches are set, and what appears to be a wall is filled into all the open spaces, until the frame is covered and hidden from sight. The outside that is seen with the eye is a mere shell, supported from floor to floor, and not even supporting its own weight. Between the new and the old we are now building every conceivable kind of construction, and the Engineer must learn to adapt himself to everyones ideas and make good buildings by any variation of method.

The ordinary procedure in the design of large fireproof buildings begins with the sketches, outlining in a very general way the arrangement of the interior of the building, and the outlines of the elevations, in the architect's office. This work is usually done with a pencil, and oftentimes it is freehand. In some cases the arrangement may be determined entirely, and quite easily, by the uses to which the building will be put, but more often it must have the study of both the architects and owners. When the lines seem to be satisfactory they are usually worked out again in pencil more carefully to scale. Before iron was used, as it is now, the interior of most large buildings was cut up by division and partition walls which were generally self-supporting from the foundations; indeed, in most cases these interior walls were required to support the floors. This made it necessary to arrange the rooms to some extent over each other, so that the partitions and the floors could have a proper support. In some cases this was quite a complicated study. With the advent of steel all interior walls and wall supports for floors became unnecessary. Rooms and apartments can be arranged in any way in the modern building, and their arrangement on a particular floor can be entirely independent of their arrangement on the floor either above or below. The arrangement of the columns is generally the first feature of construction to be studied out. The work of an Engineer should begin at this point, and this is true whether he is retained as a consulting Engineer or employed on a regular salary. The columns must be arranged so that they will not interfere with the use and finish of the rooms. It is very easy to arrange columns to suit one floor plan, but the difficulty is in finding one fixed arrangement for the columns which will be equally suitable for all the floor plans, and the architect himself must decide what is satisfactory and what is not. It may not be possible to make them all satisfactory, in which case he will probably change the arrangement of the rooms to suit the iron work; but the columns should also be arranged so that the steel frame will be economical in construction, so that the building will be stiff and strong laterally, and so that the foundations can, if possible, be simple

construction. These are engineering problems, and, perhaps, the most difficult points to decide upon. Sometimes it is necessary to figure out different arrangements to find one which will be satisfactory.

If the building requires some special provision for the lateral strength, a general plan should be studied out for it at this time, for it may also require some modification of the design. After these things are decided upon the architect can proceed with his finished drawings, and finish the general and detail drawings of the building. The steel construction should be shown on an entirely separate set of drawings, and if this work is figured out, and these drawings are made by a consulting Engineer, it can be done very easily in a separate office, and can be completed ordinarily almost as soon as the general drawings.

The method of work in my own office is very definitely fixed. A drawing is first made showing the centers of all the columns, the building and the property lines, and all the measurements relating to them, accurately to the sixteenth of an inch. A copy of this drawing is furnished to the architect for his formal approval, and all work in both offices is made to conform to it. The arrangement of floor beams is studied out in a general way, and the sizes of the important ones are calculated. A plan of each of the floors is then made on tissue paper, each showing the location and spacing of the floor beams. The exact area in square feet tributary to each column is calculated and marked on the plan. Before the size of any of the beams is finally fixed, the weight of the floor and the live load which it will carry must be determined, and approved by the architect. These weights are multiplied by the areas on the plans, and the results are tabulated until the total live load and the total dead load, which each column must carry at each floor, is recorded. When the architect has fixed the lines of the elevations the exact size and position of the window openings, and the thickness of the masonry, the calculations of the beams in the walls can be made. Generally we make only one calculation on a page of paper. At the top of the page we draw a sketch indicating the beam and the loads it carries. These loads are indicated by letters "A," "B," "C," etc. Below the sketch the calculation of the weight of each load is expressed, a separate expression for each load. Then the reactions are expressed, the figures giving the position of the loads being taken from the sketch. The wall reactions and the floor reactions are kept separate. The last expression is for the bending moment. This work is all done by one person. If the work proceeds as we prefer to have it, these expressions are examined and checked by another person before the extensions are made and the result determined. The actual

multiplying and dividing to obtain these results is destroyed, but the sheet expressing the operations to be performed is preserved. All extensions are also checked, so that finally four different men are really employed in each calculation. This would seem to be an unnecessary labor, but it is really not so great as it at first seems to be, and accuracy is insured.

A good many characteristics must be combined in a successful Engineer, but the one characteristic that is necessary above all others is accuracy. Doing things pretty nearly right is never satisfactory. The additional work of checking both the expressions and the actual calculations, cost less, in proportion as the first work is accurate. The reactions from the wall loads are kept separate from those of the floor loads, for use in calculating the column loads. When all this work is completed, the wall reactions are transcribed and scheduled in a convenient form, so that the total wall load, carried on any given column at any given floor, may be readily found.

Sometimes the architect can give sufficient information for these wall calculations, without being able to give all the data required to determine exactly what size and shape the beam should be. The calculation gives the bending moment, which, of course, must not be less than the moment of resistance of the beam, but before the section of the beam can be fixed it is necessary to know the exact size of the lintel, if any, whether it is of terra cotta or stone, or of other material, and the iron in the spandrel must be arranged to perfectly support, not only the lintel, but all the masonry that it carries. It must be so placed that it can be properly covered and fireproofed inside and out. It must be made convenient for the floor construction. It must be so arranged that it can be well, and, if possible, easily connected to the column. It must be neither too high nor too low. It must not interfere with the flues or pipes that may be required in the walls, and, indeed, the needs of every artisan employed in the construction of the building should be considered in fixing the dimensions, position, and condition of each individual spandrel.

The problem is partially an engineering one, partially an architectural one, and partially one taxing only the ingenuity of the designer. It is rarely that one man fixes and decides upon a spandrel, in our office. A pencil sketch on tissue paper is first made by some one who is familiar with the details of the construction, and this sketch is studied by all who are acquainted with the work, until it seems to us that it is properly done. These sketches are then sent to the architect, where they are again examined, and approved or disapproved. Ap-

proved sketches are blue printed and used in both offices as the basis of all further work. In our office they are traced in ink, as many as possible on one drawing. Interior floor beams used as girders, with complicated arrangement of loads, are calculated in the same way that we figure the spandrel beams. The final drawings of the floor plans, showing the size of each beam and its position in the plan, are made on tracing cloth.

The column loads are tabulated in a book which we have especially ruled for the purpose. The form which we use keeps the dead floor load and the live floor load, the masonry, the column covering, the weight of the column itself, and any special load due to machinery, tanks, etc., separate and distinct from each other. It also gives the accumulative load in the column stack at each floor level. Ordinarily we express these loads on a large drawing, which we call a column load sheet. The loads that come in the building, one over the other, are placed on the sheet one over the other in squares, a vertical space in the drawing being made for each column. We put the total in each case in the center of the square. In one corner we put the total section in square inches required for the given concentric load. In another corner we put the number of inches required on account of the eccentricity of loading. In the third corner we put the number of inches of section required on account of wind forces. In most cases this is zero, but it occasionally amounts to a good deal. In the fourth corner of the square we put the total number of inches of section of metal required. Another drawing is made of exactly the same form, which we call our column section sheet, and in the corresponding squares the metal to be used in making the column is named. If columns are to be made long enough for one piece to extend through two or more stories the drawing can be made to show this, and the section of the entire column must be large enough for the greater load. The drawings and specifications complete should indicate and describe every piece of iron in the building sufficiently in detail to permit of the work being accurately estimated and properly detailed. This is all that should be required of the architect, unless he is paid more than the ordinary fee for his work. The work that is done by the Consulting Engineer should be paid for by the architect, or at least deducted from the architect's compensation. Finally a separate drawing must be made for every piece of iron in the building, unless two pieces should be found to be entirely alike, when one drawing will answer for both.

Some of this work is done by Consulting Engineers employed by contractors, or by the owners of the building, but most of it is done by

the contractors for the structural iron. There are few architects in the country who are good Engineers, and whose judgment with reference to the structural iron work used in their buildings is entirely reliable. Some of these men do their own work, some of them employ Engineers under salary in their own offices, and others depend entirely upon the services of some Consulting Engineer whom they retain for such services when it is necessary. There is another class of architects, who are not Engineers and are not competent to determine the character of the steel frames required in the construction of their buildings, but who are yet wise enough to employ competent men, either in their own offices or outside. Unfortunately, there is a third class of architects, who know little or nothing about the strength of iron work and how it should be designed, who yet have not employed experienced counsel. Some structural iron contractor generally fixes up a set of iron drawings for them, so that it will pass muster, and the world at large knows but little difference. The owner, however, whether he knows or not, generally pays for it well. He pays the biggest price and generally gets the poorest article.

If the plans and specifications are made definite and complete when the work is estimated, nothing need to be guessed at or speculated upon, and the cost may be calculated as accurately as so many bushels of wheat when the price per bushel is known ; also if the plans are what they should be the work itself must be acceptable. There is a rapid and well defined growth in the intelligence of architects and builders along this very line. The East has seen a marvelous change in two years, a change that words can hardly describe. Improved methods of work, better drawings, better specifications, and greater care all along the line of construction is marking a new era in the building construction of the great City of New York.

It is impossible in one short hour to more than touch the subject here and there. Much has already been done to bring the treatment of the whole problem to a scientific basis, and yet there is very much more to do. It would be almost impossible to find two competent Engineers or architects who would agree exactly as to the best method to provide for the lateral stiffness of a narrow, tall building, and the most scientific treatment today will undoubtedly seem deficient before any of you are well established in business. We think we know a good deal about column designing, but there is a good deal more that we do not know, and there is probably no subject in the whole realm of structural engineering concerning which our practical knowledge is so unsatisfactory.

The foundation problem as it is applied to great buildings is another in which we have been making very rapid strides out of darkness into light, and yet we are groping. We know a whole lot more than our fathers did, indeed, more than was known ten years ago, and yet there is much concerning which we are only speculating. Steel construction by making buildings very large, has made the foundation problem correspondingly more important. By concentrating the weight of the building on columns at fixed centers it has made it more definite, and to that extent has simplified it. The interests of adjoining property add their complications. Not many years ago the architect thickened out the walls of the building at the base, and in city buildings generally on one side of the wall only, making the footing about as wide as he made it on another building that he built before, a little wider if the building was a little larger, and a little narrower if the building was a little smaller, and let it go at that. The only rule he had was some rule of thumb or precedent. Some of the leading architects were scientific, but they were really exceptions.

When the Chicago post office was built, and that is not a very old building, something extra was undertaken and the entire area of the building was covered with a layer of concrete, supposed to be thick enough to hold together under any circumstances. Immense piles of masonry were put in places and large areas were left unloaded, and gradually as the years have gone by, the big layer of concrete has moved up and down under the unequal load, and the masonry has broken, and lives have been in danger, and one after another the great court rooms were rendered unsafe, until the building has finally been condemned. The Board of Trade has been compelled to remove the top of the tower of their headquarters at the head of Lasalle St., because the unequal settlement of the building threatened its destruction. We know better than this now. We have learned that the loads should be accurately calculated, that foundations should be proportioned to the loads, that live loads and dead loads should be kept distinct that in all kinds of foundations the footing should be loaded uniformly, and if the center of the load does not coincide with the center of the resisting area some combination must be employed to accomplish the same end, and these facts hold true on the steel and concrete cassion, reaching down to the solid rock as well as on the widespread steel grillage foundation, supported on soft and yielding clay.

The allowance which should be made for the live loads on beams and columns, and foundations, too, has had as yet no satisfactory treatment. There is no uniformity in practice, there is no uniformity in

law, there is no agreement among Engineers as to what is most desirable. The present New York building law requires that the beams and the girders and all the columns in office buildings should be calculated for one hundred pounds of live load for every tributary square foot of floor area. A new law is being prepared which will probably be before the Legislature for passage a year hence, which specifies that this factor shall be sixty pounds instead of one hundred, and the amount taken upon the column shall be still less than that. In my own opinion sixty pounds of live load per square foot of tributary floor area is sufficient allowance for joist beams, provided, however, that halls, and entrances should be strong enough to take a load of four thousand pounds concentrated over a small area. I think, also, that in most floors the girder construction figured on a basis of fifty pounds per square foot will be equally sufficient. While joist beams may be fully loaded, it would be almost impossible to load to their full capacity all the joists that connect to any given girder. It is also equally true that while one beam may be loaded to its full capacity it will be almost impossible to load all the floor tributary to any given column in an ordinary office building to the same average load per sq. ft., and it will be still more unlikely that all the floors should be loaded in the same way and at the same time. On this account there is no good reason why the live load taken into the columns should not be materially reduced.

On these same accounts, the live load which actually goes into the foundation, and which is carried by it in the ordinary office building or hotel, or in any high building with many floors, is really only a small percentage of the amount which we would obtain by multiplying all the floor areas tributary to the column on all the different floors, by the same factor which should be used to calculate the strength of a single beam. More than this, if the foundation is a yielding one this small portion of live load should by all means be entirely eliminated. It has been practically demonstrated with the yielding foundations in Chicago, that the settlement of different columns in a building is in proportion to the positive fixed load which they carry, and so far as the live load represents a moving load it should be omitted in the calculations of footing areas.

Some very interesting experiments were recently made in New York to show the effect on a floor of a moving crowd of people. These experiments were made by Mr. Geo. B. Post, and the Superintendent of Buildings, at the shops of J. B. & J. M. Cornell, and I am able to describe them to you through the courtesy of Mr. Post. Ten men were selected having an average weight of one hundred and seventy-nine

pounds, and the tests were made on a large platform scale. When standing as closely together as men conveniently can stand on an elevator, they weighed about one hundred and three pounds per square foot of floor covered. Subsequent experiments, however, showed that the same men could be packed closely enough together to increase this to a possible maximum of 120 pounds per square foot. Walking over the platform in step, the total load on the scale beam was increased only ten pounds. Running over the platform added nothing more to the expressed weight. Taking plenty of room on the scale for individual motion, each man jumping as high as he could jump, was also scarcely noticed on the scale beam. This would seem to indicate that columns carrying floors where assemblies may congregate need not be increased because of moving or dancing. It would also indicate that 100 pounds per square foot of live load is ample for the calculation of the floor beams, and that the laws governing the vibration of bridges under a moving load could not be applied to building construction.

Possibly I should not conclude without a word in regard to the future of this particular branch of engineering work. It seems to me to be in its infancy. The new buildings are in every respect so much better than the old, that not only will vacant property be built with steel construction, but the old buildings, especially in the large cities of the East, will be rebuilt and are being rebuilt, the modern taking the place of the old. In five years the outlines of lower New York have been changed. Probably $60,000,000 will be spent in 1896 in house building, and a very large portion of this sum will be large buildings employing steel in their construction. The old buildings will be torn down over a large part of the lower city before you are middle aged, and the new construction will take its place. Mr. E. L. Corthell, after a very careful and elaborate study of the population of great cities, says that he has the temerity to suggest that New York will have a population in 1920 of between six and seven millions, and that Chicago may have as many as eight millions. The amount of work that must be done in the aggregate to build such cities is beyond our comprehension.

BLOCK SIGNALLING.

NOTES ON A LECTURE DELIVERED BY GEO. W. BLODGETT.

Block-signalling is a subject whose importance is as yet unrecognized, the problem being one only known to technical students, engineers and technical editors.

The need of block-signals was made apparent with the extension of the railroad, in fact nearly all the systems of signalling are as old as the railroads themselves. George Stephenson used first a candle in the window of a conveniently located house for his signal to a train ; when the light was exhibited the train was to stop, otherwise not.

The development of systems of block-signalling has been very slow. The systems of signalling are the :

(a) Absolute block, allowing only one train in a section at a time.

(b) Permissive block, allowing more than one train in the section or block at a time, under certain restrictions.

The absolute block system is in very general use in England. For signals we have a tall post with a pivoted arm which may be given either a nearly vertical position or a horizontal one. In this system, designating the blocks or sections as A, B, C, and D, the operator at B asks the operator at C for permission to send a train past B into the block B-C. When C gives permission B raises his signal allowing the train to pass and immediately thereafter drops his signal to the horizontal "danger" position. Before the train gets to C, the latter asks D for permission to send the train into the block C-D. If all is right, C has his signal at safety when the train arrives and no loss of time results, the train going by without stop.

In England and on the Continent, this absolute system is almost universal. There is nothing in this system to prevent a careless man neglecting his signal and making mistakes, hence the natural invention of a lock system.

The Sykes lock and block signal, having at each man's signal lever an electric lock which (lock at signal B) is unlocked by C if train has passed into block C-D, and C cannot unlock B's signal (of danger)

until this has taken place, and B cannot move his lever giving a train the safety signal into block B-C, until C unlocks his lever. The safety by this system has been largely increased as it takes three men to get a train through a section, and two men to change a signal. This system is in use on the N. Y. Central from Albany to Buffalo. It is an expensive system to operate and to maintain, as men are required day and night in the signal towers; thus only the most prosperous roads can use it.

RUNNING TRAINS BY TELEGRAPH.

This system originated on the Erie railroad at Elmira. The road had been previously run on schedule, absolutely no deviations from which were allowed. This made the delays due to late trains very vexatious as well as costly. At Elmira an express from the east was four hours late, with a stock train at that point waiting to go west, and another at Corning waiting to come east. The assistant superintendent, on his own responsibility, defied the schedule, and by telegraph sent the stock train to Corning, and the second was brought to Elmira before the express arrived. The idea at the time was a very novel one but instantly grew into favor, and now some roads are run almost exclusively by telegraph, while some still stick to schedule running.

In the telegraph system, the road is divided into divisions of from 100 to 125 miles, and these into sections of from two to ten miles, at each of which is an operator. For each division there is an expert operator in absolute control of all trains of the division. He is the train despatcher. He has a record kept of the arrival and departure of all trains. He writes his orders in triplicate—one to the engine man, one to the conductor, and one to be kept for record. On the best roads the orders are verbally repeated back to the despatcher.

The Pennsylvania railroad uses partly block and partly telegraph system. In their system less telegraphing is required than in some of the other systems, but at its best it is a very expensive system.

Many attempts have been made to get an automatic machine system, that the train itself may change the signal in the block just left. Lightning has furnished the current in some electric systems that has caused the wrong signals to be shown with fatal results.

The rails of the track are very generally made use of for the circuit of electric-automatic systems. Sections of track, a mile or less in length, are insulated from one another by wood fibre at the ends of rails and around the bolts through the fish plates. At one end of the section there is a battery connected one pole with each rail, and at the

other end of the section each rail is connected with a pole of the signalling apparatus, so that when there is no train in the section the current flows and the signal indicates safety. When the train enters the section, the current flows through the wheels and axles, short-circuiting the signal, which then drops to danger. A small current is used with a relay and stronger battery at the signal end. A broken rail will cause the signal to drop to danger, and the sections are made to overlap to prevent danger from a train's breaking down in the section, or at its beginning.

The signals in this system cost about $500, and for yearly maintenance about $100, whereas by the manual system of signal towers and operators, the salaries of the latter are alone $400 to $800 per year per tower, besides the maintenance. Thus the electric-automatic system is seven or eight times cheaper.

SIGNALLING IN YARDS.

At junctions and terminal stations interlocking signals are widely used. Interlocking saves space and time, since several systems or combinations of movements can be safely carried on at once. Greater safety and freedom from blunders are insured, as two routes from yard to train-shed cannot be opened at once. The levers for the movement of the various switches of the yard are brought together in the tower. At the Grand Central station in New York there are 100 levers side by side. A special system of levers are, of course, worked out for each yard, and when the first of a series of movements with the levers is made, the operation unlocks the second lever of the combination, which operation in turn locks the first lever so that it cannot be displaced. The last lever of the series gives the signal to the train to proceed. The train must make the movement the signal indicates. Thus the operator in the tower controls the placing of the empty cars in the shed, also of the full trains and the movements of loaded trains outbound, as well as the removal of the empty coaches and all switching of engines or cars from track to track.

In adjusting an interlocking system to a yard, the first thing usually done is a rearrangement of the tracks. The old system of switches from one track to another requiring a great deal of space is replaced by "slip" switches in ladders. In this system, one track crosses all the tracks in the yard, with short "slip" switches into every track crossed. These ladders, crossing the yard in two directions, facilitate passing any obstruction on the tracks; also the shipping, rapidly and within

short space, of empties, and the transference of cars from one train on one track to another on a different track.

Interlocking at a junction promotes safety at that point to a great degree. It prevents a branch train from running out on to the main track when the signal for the latter indicates safety to the main line trains. A signal is usually in position when a train approaches a block, the operator having been notified of the approach of the train from some near by station.

In the electric locking device the current is usually furnished by storage batteries or by "gravity" cells, it being desired to have it as constant as possible. Often the power to move the levers or the switches in the yard is furnished by compressed air. This system combined with the electric is in use at the Pittsburg terminal of the Pennsylvania railroad. In the signal towers, by means of a model of the various tracks and switches in the yard, the operator can see at a glance what movements have been made by the levers, all operations being repeated on the model before him.

STRESSES AND DEFLECTIONS IN CIRCULAR RINGS UNDER VARIOUS CONDITIONS OF LOADING.*

BY CLAUDE W. L. FILKINS, M. C. E., AND EDWIN J. FORT, M. C. E.

In the Journal of the Association of Engineering Societies for December, 1893, Professor Benjamin of the Case School of Applied Science has endeavored to adapt Gordon's formula for columns to the case of a circular hoop under load, and has given the results of some carefully made experiments upon hoopsloaded at the crown with a single concentrated load. These results which he has kindly furnished us are given on Plate 3. Starting from the results that we have obtained, Mr. Wm. H. Searles, a Civil Engineer of Cleveland, has attempted to extend their application to a perfectly elastic ring. The results of his work are embodied in a paper published in the journal of the Association of Engineering Societies for September 1895.

I :—In the case of a hoop bearing a concentrated load ($=P$) at its crown, we may assume that the forces acting at any section may be resolved into a "stress couple," a "shear," and a "thrust," just as in many cases of loaded beams. As the distortion of the hoop is not supposed to be great, the material not being strained beyond the elastic limit, we use in the computation the dimensions of the hoop before distortion. This may not materially affect the results, because the change of the diameter is small compared with the length of the diameter itself. This approximation, of course, destroys the mathematical accuracy of the results obtained, but the use of the true diameter in its strained con-

*The solution of this problem for the case of equal and opposite forces applied at the extremities of a diameter is also to be found in "Constructions Métalliques" by J. Resal, and in "Résistance des Materiaux" by A. Madamet. The theory of the flexure of curved pieces, of which theory this problem is a particular example, is to be found in Castigliano's "Théorie de l'Équilibre des Systèmes Élastiques," p. 264 et seq. The general formulae are there derived by a simple application of the principle of "least work."—ED.

We wish to acknowledge our appreciation of the timely hints and the thorough review of this article made by Mr. C. W. Comstock of Cornell University.

dition would complicate the problem, and the additional accuracy obtained might not be worth the trouble. The radius used in the solution of these problems is the inner radius of the hoop plus one-half the thickness of the hoop. In the first case, we neglect the weight of the hoop, and proceed to find the shear ($=J$), the thrust ($=T$), and the moment of the stress couple ($=M$) at the points A, B, C, and D, see Fig. (1). Presumably these four points are the points of greatest stress and are, therefore, the ones to be particularly investigated.

Take any small portion of the hoop, of length $= ds$. see Fig. (2), as a free body, and consider the forces acting at the two sections H and K of this element. Let the change in the angle be tween the tangents at A and E, due to the load, be $d\phi$ the original angle between these tangents being $= d\theta$.

K, H, G, W, is the unstrained form of the element, $EA = ds =$ original length of all the fibre lines (very nearly, for large radius of hoop). $d\lambda_1 =$ the distance by which all fibres are shortened by the thrust. Under the influence of the thrust and the stress couple the position of the tangent at A has changed to DA' and the section HG has changed to $H'G$. $d\theta$ now $= d\theta \div d\phi$. We may find $d\phi$ as follows :—

The shortening of the fibre ($= d\lambda_2$) at a distance e from the neutral exis is due to the force $P_2\, dF$ and therefore $d\lambda_2 = \frac{P_2\, ds}{E}$ (from the definition of $E =$ modulus of elasticity of the material, *i. e*,

$$E = \frac{(\text{stress per unit area})}{(\text{relative distortion})} = \frac{p}{(d\lambda_2 \div ds)}\text{; therefore } d\lambda_2 = \frac{p_2\, ds.}{E}$$

But from Fig. (2) $d\lambda_2 = e\,d\phi$; therefore $e\,d\phi = \frac{p_2\, ds.}{E}$

The usual formula for M, $=$ the moment of the stress couple, is $M = \frac{p\, I,}{e}$ where $I =$ moment of inertia of the section, $p =$ the unit stress upon the outer fibre, and $e =$ distance from the neutral axis to the outer fibre. Therefore, substituting in the above equation, we obtain

$$d\phi = \frac{M\, ds}{EI} \qquad (1)$$

which is the formula required. Evidently the change of angle between the ends of a finite length of the hoop $= \int_0^\phi d\phi$ where $\phi =$ the angle included between the radii drawn to these two points.

Referring to *Fig. 3*, let OC be the unstrained condition of a portion of the hoop. Take O as an origin, the co-ordinates of any point on

the hoop being x and y, the axis X being arbitrary in direction. The other dimensions are as shown in the figure. $O_n E'' D' C_1 C$ is the strained position of a portion of the hoop.

Suppose C and C's tangent to remain unchanged in position. Suppose also each ds to bend through an elementary angle $= d\phi$. The bending of the first ds causes the whole finite piece $O C_1$ to turn about C_1 as a centre through the angle $d\phi$, and O moves to O' through the arc $dv = u\, d\phi$. Conceive each ds to be thus bent through its angle $d\phi$ and the final position of O will be at O_n. Then the total displacement of O along the axis

$$X = \triangle X = \int_0^c \delta x, \text{ and } \triangle Y,$$

the total displacement of O along the axis $Y, = \int_0^c \delta y$. But $dv = u\, d\phi$, and from similar right triangles $dx : dv :: y : u$ and $dy : dv :: x : u$; therefore $dx = y\, d\phi$ and $dy = x\, d\phi$.

$$\therefore \triangle X = \int_0^c \delta x = \int_0^c y\, d\phi = \int_0^c \frac{M y\, ds}{EI}; \qquad (2)$$

thus giving the projection on X of O's displacement relatively to C and C's tangent. Also

$$\triangle y = \int_0^c \delta y = \int_0^c x\, d\phi = \int_0^c \frac{M x\, ds}{EI}, \qquad (3)$$

giving the projection on Y of O's displacement relatively to C and C's tangent.

We now have three perfectly general equations which are applicable to any condition of loading, since the actual loading has not entered into their determination.

A fuller demonstration of the above three formulæ for the more general case of arch ribs may be found on pages 445, 448 and 449 of Church's Mechanics of Engineering, from which the above demonstration has been taken almost verbatim.

These formulæ are homogeneous; that is, any system of units may be used in applying them. In the following solutions, E and I have been taken as constants, but should they in any case be variable, they can be treated in the same operation if they vary according to a function of the general dimensions of the hoop. These formulæ can be applied to girders and hoops only as far as the material is continuous.

In case I, where a concentrated load is placed at the crown of the hoop, the weight being neglected, there is a sinking at the crown, and

a bulging at the extremities of the horizontal diameter, the distorted hoop being perfectly symmetrical about a vertical and about a horizontal axis. We thus see that each quadrant, from the crown or bottom to the horizontal diameter on each side, is under the same conditions of stress and strain. Therefore no horizontal displacement has occurred at B and D, due to the application of the load. Also, the tangents at these two points have remained horizontal. The tangents at the points A and C have remained vertical and one has not moved vertically with reference to the other; therefore,

$$\int_D^B \frac{M\,ds}{EI} = \int_D^B d\phi = O =$$

the whole change in the angle of the tangents at these two points. Also

$$\int_D^B \frac{My\,ds}{EI} = \int_D^B dx = O =$$

the whole horizontal displacement of B with reference to D. Also

$$\int_D^A \frac{M\,ds}{EI} = O.$$

To find the thrust at B and D consider the free body shown in *Fig. 4*. From the symmetry of the loading $T_B = T_D$, both being compression, or both tension, but for equilibrium $T_B + T_D = O$. Therefore $T_B = T_D = O$ and $M_B = M_D$. Also, for equilibrium, $J_D - J_B = O$, or $J_B = J_D$. But from *Fig. 5* $J_B = P/2$. Therefore $J_B = J_D = P/2$.

Again, to find the thrust and shear at A and C, consider the free body in *Fig. 6* between A and B. For eqiulibrium,

$$J_A - T_B = O; \text{ therefore } J_A = O$$
$$T_A - J_B = O; \text{ therefore } T_A = P/2$$

$$M_A + M_B - J_B r = O; \text{ therefore } M_B = J_B r - M_A = \frac{Pr}{2} - M_A.$$

Now imagine the portion of the hoop shown in *Fig. 7* as a free body and consider the forces acting upon it. The section at A will be acted upon by the thrust T_A and the moment of the stress couple $= M_A$; ($J_A = O$). The section at O (any section) is acted

upon by M, J, and T, all unknown. Take O for a centre of moments; then $M = \frac{Px}{2} - M_A$. Substituting this value of M,

$$\int_D^A (Px/2 - M_A)\,ds = O, \text{ or } \int_D^A \frac{Px\,ds}{2} = \int_D^A M_A\,ds.$$

But

$$ds = \sqrt{dx^2 + dy^2} \text{ and } dy = \frac{(r-x)\,dx}{\sqrt{2rx - x^2}}, \text{ or } ds = \frac{r\,dx}{\sqrt{2rx - x^2}}$$

$$\therefore \frac{Pr}{2}\int_0^r \frac{x\,dx}{\sqrt{2rx - x^2}} = M_A r \int_0^r \frac{dx}{\sqrt{2rx - x^2}}$$

Solving the above equation, we find

$$M_A = \frac{Pr}{2}\left(1 - \frac{2}{\pi}\right) = .18169\,Pr. \qquad (4)$$

This also $= M_C$, from the similarity of conditions at the two points.

$$\therefore M_B = \frac{Pr}{2} - \frac{Pr}{2}\left(1 - \frac{2}{\pi}\right) = \frac{Pr}{\pi} = .31831\,Pr \qquad (5)$$

We have now obtained the following values for the forces acting at the four points A, B, C, and D.

$$M_B = \frac{Pr}{\pi} = .31831Pr = M_D$$

$$T_B = O = T_D$$

$$J_B = P/2 = J_D$$

$$M_A = .18169\,Pr = M_C$$

$$T_A = P/2 = T_C$$

$$J_A = O = J_C$$

To find the horizontal displacement $(= \Delta x)$ of the point A of the hoop, consider the portion of the hoop shown in *Fig.* (*8*) as a free body with the forces acting at the points A and O. Between the two points A and B the equation $\Delta x EI = \int_B^A Myds$ is true, M and y being as shown in the figure. Taking the centre of moments at the neutral axis of the section at O, we have (since $J_A = O$)

$$M = \frac{Px}{2} - M_A;\ \Delta x E I = \int_B^A \frac{Px}{2} y\,ds - \int_B^A M_A y\,ds;$$

$$y = \sqrt{2rx - x^2};\ dy = \left(\frac{r - x}{\sqrt{2rx - x^2}} dx\right);$$

$$\text{and } ds = \sqrt{dx^2 + dy^2}.$$

Substituting these values in the above equation and solving for Δx we obtain

$$\Delta x = \frac{Pr^3}{4EI}\left(\frac{4}{\pi} - 1\right) = .06831 \frac{Pr^3}{EI} \tag{6}$$

But the total elongation of the horizontal diamenter is

$$2 \Delta x = .1366 \frac{Pr^3}{EI} \tag{7}$$

This expression is the one to be used in comparison with the values obtained by experiment.

Now, in order to find the shortening of the vertical diameter due to the concentrated load, we observe that the tangents at the points B and A have retained their original direction. Also between these two points the equation $\Delta y EI = \int_B^A Mx\,ds$ is true. Now consider the portion of the hoop shown in *Fig.* (9) as a free body. By taking the centre of moments at the neutral axis of the section at O, we obtain

$$M = J_B x - M_B;\ \text{also } x = r \sin\phi \text{ and } ds = r\,d\phi$$

$$\text{Therefore } EI \Delta y = \int_0^{\frac{\pi}{2}} (J_B r \sin\phi - M_B) r^2 \sin\phi\, d\phi$$

Solving this expression for Δy, we obtain

$$\Delta y = \frac{Pr^3}{2EI} \frac{(\pi^2 - 8)}{4\pi} = .0744 \frac{Pr^3}{EI} \tag{8}$$

But the total shortening of the diameter is

$$2 \Delta y = .149 \frac{Pr^3}{EI} \tag{9}$$

This is the value to be used in comparison with that obtained by experiment for the shortening of the vertical diameter of hoops under concentrated loads at the crown.

It will be noticed that these two expressions, giving the lengthening of the horizontal, and the shortening of the vertical diameter, are perfectly consistent and indicate that the amount of this distortion varies directly as the amount of the load P and the cube of the radius of the hoop, and inversely as the modulus of elasticity (E) and as the dimensions of the cross section, or as the moment of inertia of the section; and that this distortion is independent of any other quantities. Also the stresses in the hoop hoop due to this load P depend only upon the load P and the radius r, as we should naturally expect.

In Professor Benjamin's paper, it was stated that in every instance, the hoops tested failed at the extremities of the horizontal diameter. This would indicate a maximum unit stress at A and C, that is $p_A > p_B$ or $\frac{6hPr}{bt^2} + \frac{P}{bt} > \frac{6kPr}{bt^2}$ where $h = .18169$ and $k = .31831$. Whence we obtain $\frac{t}{r} > 6\ (k - h)$ or > 0.81972. From this we conclude that the hoop must almost be a solid circular disk in order that the unit stress at A exceed that at B. Not knowing the distribution of stresses in such a disk, or in a hoop approaching such a disk, we must look for the theoretical maximum at some other point. Combining equation (12) and the first of (14) below, we obtain a discontinuous equation for the unit stress for any angle ϕ and any ratio of $t:r$. By taking values of ϕ between $O°$ and $90°$, we find by comparison, that the maximum unit stress occurs at B for all values of $t:r$ less than 0.81972. Thus

$$p_B = \frac{6kPr}{bt^2} = 1.90986\,\frac{Pr}{bt^2}. \qquad (10)$$

By introducing the safe unit stress, and given data into this equation, we may derive the unknown parts in design.

That the maximum stress does occur at A and C seems to be demonstrated experimentally. That theory does not so indicate it, is probably owing to the neglect of the distortion. As the load becomes greater, and as the horizontal diameter becomes more and more elongated, the moment M_A becomes greater than that indicated by theory, and the stresses due to this moment are correspondingly increased. Initial stress in the hoops tested, may also partially account for the results obtained.

Let us now consider the expression for the moment at any point of the hoop.

$$M = J_B x - M_B \left[\text{See } Fig. \ (9) \right] \tag{11}$$

$$\text{Where } x = r \sin\phi$$

$$J_B = P/2$$

$$M_B = Pr/\pi$$

$$\text{Therefore } M = Pr \left(\frac{\sin\phi}{2} - \frac{1}{\pi} \right) \tag{12}$$

This may be placed in the form A ($\sin\phi - B$) which is the equation of a curve of sines. By giving the proper values to P, r and ϕ we may plat the moment diagram for the hoop, shown on *Fig.* (*10*).

If now we place $M = 0$ and solve for ϕ in the above equation, we obtain the point of zero moment on the hoop. Sin ϕ for this point $= .63662$ or

$$\phi = 39^\circ \ 32' \ 25'' \tag{13}$$

At this point on the semi-circle a horizontal line was drawn as a reference line from which to measure moments. The construction of this diagram is evident upon inspection of *Fig.* (*10*).

By placing the vertical and horizontal components of the forces acting in *Fig.* (*9*) $= o$ and solving for T and J we obtain

$$\left. \begin{array}{l} T = J_B \sin\phi \\ J = J_B \cos\phi \end{array} \right\} \tag{14}$$

From these two equations we may plat the thrust and shear diagrams shown in *Fig.* (*10*). Evidently the thrust diagram is a simple curve of sines, and the shear diagram a curve of cosines. These diagrams are not platted to the same scale and do not give the relative porportions of the moment, thrust, and shear at any point. The straight horizontal line from which the thrusts and shears are platted is equal in length to the semi-circumference of the hoop and the diagrams for the other half of the hoop would be a repetition of those shown. It is noticeable that, as in the case of a loaded beam, the shear passes through zero at the point where the moment is a maximum, or at least a local maximum.

A very easy method of drawing the moment diagram is to add the moments at A and B and to assume that the radius of the circle, from

which the diagram is drawn, is equal to this amount by any desired scale. The curve of sines for this circle will then give the moment of the stress couple, measured by the same scale from the proper axis, at any point of the hoop.

The equations giving the change in length of the vertical and horizontal diameters are equations of straight lines. On Plate III, these equations have been platted for the seven hoops tested by Professor Benjamin. The values given by the formula are shown by dotted lines. The corresponding values obtained experimentally by Prof. Benjamin are shown by full lines.

In almost every case the values given by the formula are smaller than those given by experiment, but where the latter approximate closely to a straight line, there is a practical coincidence between the results. This is particularly noticeable in hoop No. 4, where the deflections were found to vary almost exactly as the load applied, up to the elastic limit. The deflections as platted are magnified about two and one-half times, and the discrepancies between the results of theory and of experiment are thus two and one-half times smaller than appears at first sight from the Plate. We doubt very much whether the results of experiment upon structural iron of any kind if conducted with the amount of care bestowed upon these, would show a much closer agreement with the formulae by which the dimensions of structural iron are commonly computed.

Initial tension in the hoop, and increase of the moment caused by the distortion of the hoop, would both serve to account for the fact that theory gives distortions smaller than those which actually occur.

A lack of homogeneity of the material, or some defect in welding, may serve to account for the fact that in hoops Nos. 2 and 7 theory gives results somewhat larger than those obtained by experiment. Otherwise we leave this fact unaccounted for.

Gordon's formula for the strength of columns which formed the basis of Professor Benjamin's deductions, does not seem to be perfectly applicable to the case of a loaded hoop inasmuch as no account is taken therein of the stresses at the crown and at the bottom, which are comparatively large, and which exert a correspondingly important influence upon the deflection and the stresses at other points.

As a foundation upon which to build a purely empirical formula, it undoubtedly serves a good purpose, as it has some analogy to the facts.

II :–The problem presented by the hoop bearing its own weight is capable of solution by the same formulae used in solving that of the hoop bearing a concentrated load at the crown, and leads to results quite as simple and as easily applied.

The forces acting upon the hoop as a whole will be understood from *Fig.* (*11*) Plate 1; and it will also be readily understood that at the points A and C the tangents to the neutral line of the hoop are not vertical. The hoop in this case perhaps bears, as Mr. Searles has suggested, somewhat the same relation to the hoop under the concentrated load as does the beam bearing its own weight to the beam under a concentrated load at the centre, but as the results will show, the comparison is not very close.

If we imagine the hoop cut at B and close on the left of D and if we supply the forces acting at the sections of these points, we obtain a system of forces in equilibrium as shown in *Fig.* (*12*). Placing the horizontal components of these forces $=0$, we obtain

$$T_B = T_D$$

Now take as a free body that portion of the hoop included between B and the points adjacent to B on each side as in *Fig.* (*13*), and consider all the forces acting upon this body. Place the vertical comcomponents of these forces $=0$ and it is evident that $2J = p\,ds =$ the weight of a very small portion of the hoop and that, as the sections approach B, $J_B = 0$.

Now in *Fig.* (*12*), take the centre of moments at the neutral axis at D, and since $x = r \sin\phi$

$$M_D = M_B - 2\,T_B\,r + \int_0^\pi r \sin\phi \, dp$$

$d\,p = p\,(d\,s)$ where $p =$ the weight of unit length of hoop $= p\,r\,d\,\phi$, since $d\,s = r\,d\,\phi$.

$$\therefore\ M_D = M_B - 2\,T_B\,r + \int_0^\pi pr^2 \sin\phi\, d\phi,$$

$$\text{or, } M_D = M_B + 2pr^2 - 2\,T_B\,r.$$

Now consider as a free body that portion of the hoop shown in Fig. (14), and regard the forces acting upon it.

$$EI\Delta y = \int M x\, ds\,; \qquad ds = rd\,\phi = rd\,\theta\,;$$

$$-M = M_B - T_B y - \int_0^B p\,(ds)\,x'\,;$$

$$pds = prd\,\phi = prd\,\theta$$
$$x' = r(\sin\phi - \sin\theta)$$
$$y = r(1 - \cos\phi).$$

Therefore, $-M = M_B - T_B r(1-\cos\phi) - \int_0^\phi pr^2 (\sin \phi - \sin \theta)\, d\theta$
$= M_B - T_B r + T_B r\cos\phi - pr^2(\phi\sin\phi + \cos\phi - 1)$.

Therefore, $EI\Delta y = -\int_0^\pi [(M_B r^2 - T_B r^3 + pr^4)\sin\phi\, d\phi +$
$(Tr^3 - pr^4)\sin\phi\cos\phi\, d\phi - pr^4\phi\sin^2\phi\, d\phi]$.

By solving this expression we obtain

$$EI\Delta y = -2r^2(M_B - T_B r) + \frac{pr^4\pi^2}{4} - 2pr^4.$$

$$\text{Also, } EI\Delta x = \int_D^B M(ds)\, y = 0,$$

since B has not moved horizontally, and since its tangent has remained horizontal.

$$y = r(1 - \cos\phi)\,;\quad ds = rd\phi.$$

M is the same as before.

$$\therefore\ EI\Delta x = r\int_0^{\pi r} Mds - r^2\int_0^\pi M\cos\phi\, d\phi.$$

By substituting the value of M and solving

$$-\int_0^\pi M\cos\phi\, d\phi = \int_0^\pi [(M_B - T_B r + pr^2)\cos\phi\, d\phi +$$

$$(T_B r - pr^2)\cos^2\phi\, d\phi - pr^2\phi\sin\phi\cos\phi\, d\phi]\,;$$

$$\text{or, } \int_D^B My\,(ds) = \frac{r^3\pi}{2}\left\{ T_B - \frac{pr}{2}\right\} = 0\ \therefore\ T_B = \frac{pr}{2} = T_D$$

$$\text{Now, } -\int_D^B Mds = 0 = \int_0^\pi [M_B r d\phi - T_B r^2(d\phi - \cos\phi\, d\phi)$$

$$-pr^2(\phi\sin\phi\cdot d\phi + \cos\phi\, d\phi - d\phi)].$$

Whence, $M_B = T_B r$

$\therefore\ M_B = \frac{1}{2} pr^2$

and from above, $M_D = \frac{3}{2} pr^2$.

Evidently the shear at $D(=J_D) =$ the weight of one-half of the hoop $= \pi pr$. By substituting the values of M_B and T_B above, we obtain the shortening of the vertical diameter.

$$\Delta y = \frac{pr^4}{EI}\left(\frac{\pi^2}{4} - 2\right) = .4674\frac{pr^4}{EI}.$$

To find the lengthening of the horizontal diameter of the hoop, place $EI\Delta x = \int_A^B M(ds)y$.

Substituting the values of M, ds and y, we obtain

$$-EI\Delta x = \int_0^{\frac{\pi}{2}} [(M_B - T_B r + pr^2)\, r^2\, (1 - \cos\phi)\, d\phi$$
$$+ (T_B\, r - pr^2)\, r^3\, (\cos\phi - \cos^2\phi)\, d\,\phi$$
$$- pr^4 (1 - \cos\phi)\, \phi \sin\phi\, d\phi].$$

Whence, $-\Delta x = \frac{pr^4}{EI}(\tfrac{3}{4}\pi - 2.5) = -.1438\frac{pr^4}{EI}$.

The whole change in the length of the horizontal diameter is

$$2\Delta x = .2876 \cdot \frac{pr^4}{EI}.$$

These values of Δx and Δy were sometime ago checked approximately by experiment in the laboratory of the College of Civil Engineering at Cornell University by Professor Church, but the exact data obtained are not available for comparison.

Now, in order to find the value of ϕ where the moment is a maximum, place $\frac{dM}{d\phi} = 0$, whence $- T_B\, r \sin\phi - pr^2 (\sin\phi + \phi\cos\phi) + pr^2 \sin\phi = 0$; whence, $\phi\cot\phi = -\tfrac{1}{2}$, or $-2\phi = \tan\phi$.

By trial, we find,

$$\phi = 105^\circ\, 13'\, 45.''4\ ;\ \text{also, } \phi = 0.$$

To find the amount of this moment, substitute the first value of Φ in the expression for the moment,

$$-M = M_B - T_B r(1 - \cos\phi) - pr^2(\phi\sin\phi + \cos\phi - 1).$$

Whence, $M = .64076\, pr^2$.

When $\Phi = 0$, the maximum is $M_B = .500\, pr^2$.

It is evident that at some point between A and B, and also between A and D, the moment of the stress couple changes sign, *i. e.*, passes through zero. To determine these points place M (above) $= 0$ and solve for Φ.

Whence, $\qquad \Phi\sin\Phi + \tfrac{1}{2}\cos\Phi = 1,$

or, $\qquad \Phi = 50^\circ\, 36'\, 45''.$

Also, $\qquad \Phi = 146^\circ\, 19'\, 25''.$

In Fig. (15), place the vertical components of the acting forces $=0$. The weight of this portion of the hoop $= pr\Phi = J\cos\Phi + T\sin\Phi$. Placing the horizontal components $=0$

$$\tfrac{1}{2}pr = T_B = J\sin\Phi - T\cos\Phi.$$

Whence,
$$J = pr(\Phi\cos\Phi + \tfrac{1}{2}\sin\Phi).$$
$$T = pr(\Phi\sin\Phi - \tfrac{1}{2}\cos\Phi).$$

Also,
$$M = pr^2(\Phi\sin\Phi + \tfrac{1}{2}\cos\Phi - 1).$$

To determine the point where J passes through zero, place the above expression for $J=0$, and obtain $-2\Phi = \tan\Phi$.

Whence, $\Phi = 105^\circ\, 13'\, 45.4''$ for $J=0$.

Also, $\Phi = 0^\circ$.

This is as we should expect, namely, the shear passes through zero at the points of maximum moment, or at least at points having local maximum moments.

From the above three equations, giving J, T, and M, at any point of the hoop, the moment, shear and thrust diagrams shown on Fig. (16) were platted. These diagrams are not drawn to the same scale, and are not intended to show in their relative magnitude the stresses due to thrust, shear and bending moment.

It is noticeable that by a consideration of the above equations it may be shown that, as in the general case of bending, the first differential co-efficient of the moment with regard to the length of the hoop $\left(i.\,e.,\ \frac{dM}{ds}\right)$ is equal to the shear.

The maximum fibre stress is at D and occurs on the outside of the hoop. It is of course a compressive stress, and we should expect the hoop to fail at this point long before the stresses at any other point had reached the elastic limit.

Knowing the amount of moment, thrust and shear at D, and the modulus of elasticity of the material, we may easily compute the necessary thickness of metal to prevent undue distortion or total collapse of the hoop or tube under its own weight.

Although the three formulae used as the basis of the above solutions are perfectly general, and are those commonly used in all problems relating to arch ribs whether circular in form or not, it is probable that they would fail to apply as soon as the thickness of the metal became large in proportion to the radius of the hoop, because some uncertainty would then exist as to the manner in in which the stresses would be transmitted.

III—Another of the many cases to which the formulae derived in the first part of the article can be applied is that of a submerged intake, or water main, or other circular conduit passing through a body of water. If at any time the conduit were to become empty it would be subjected to hydraulic pressure on the outside, the stress per unit area varying directly as the depth. Thus, for a conduit of considerable diameter the pressure on different portions of the conduit is variable, and in case of great diameter combined with small weight of pipe, it becomes necessary to anchor the structure. Supposing that this is accomplished by a concentrated load at the crown and that the consideration of the weight of the conduit is neglected, we solve the problem for the acting forces, which are as shown in Fig. (17). The concentrated load at the crown is assumed greater than the difference between the weight of the conduit and the buoyant force, causing a consequent reaction at the bottom. This makes this case particularly general for similar problems, and the results obtained would be very similar to those obtained in the case of a penstock containing water under pressure.

By methods similar to those already used,

$$J_B = \frac{P'}{2}$$

$$T_B = \gamma r \left(h + \frac{3r}{4}\right)$$

$$M_B = \frac{\gamma r^3}{4} - \frac{P'r}{\pi}$$

$$J_D = \frac{P'}{2} - \frac{\gamma r^2 \pi}{2}$$

$$T_D = \gamma r \left(h + \frac{5r}{4}\right)$$

$$M_D = \frac{3\gamma r^3}{4} - \frac{P'r}{\pi}$$

h = depth of water above the crown, and γ = weight of a unit volume of water.

The deflections and the other stresses and moments can be reduced to like forms.

It is probable that, by the application of the same general formula, the stresses in the wheels of vehicles, in curved steel dams, in empty spheres, or in torpedo-shaped bodies, may be determined.

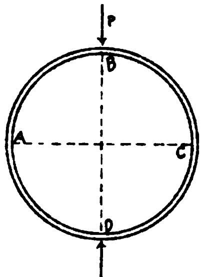


Fig.1

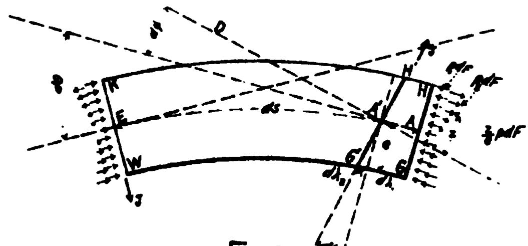


Fig.2

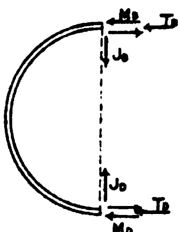


Fig.4

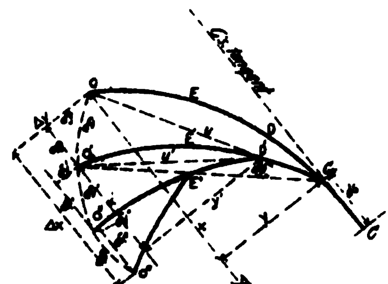


Fig.3

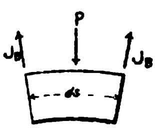


Fig.5

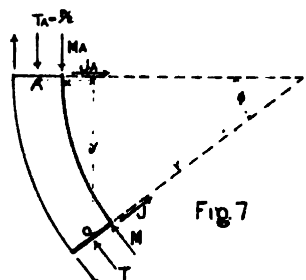


Fig.7

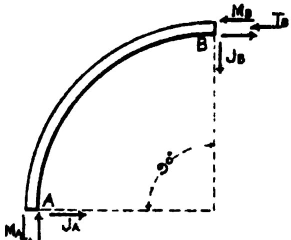


Fig.6

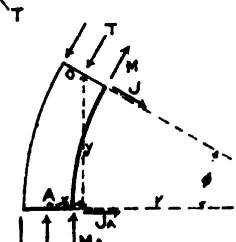


Fig.8

ON THE FLOW OF WATER IN BRANCHING PIPES.

A. L. COLSTEN AND R. H. KEAYS.

In the college year of '95-'96 the following experiments on branching pipes were made in the hydraulic laboratory of the College of Civil Engineering of Cornell University.

The object of the experiments was the determination of the relative quantities of water carried by each of two branching pipes when the angle α between them is varied—one "branch" being the continuation of the main pipe. The coefficient for loss of head at the junction of the branches with the main pipe was also determined for both branches.

The pipes were of wrought iron—new and clean.

The frictional resistance for the different velocities was neglected, being extremely small.

ARRANGEMENT OF PIPES.

The pipes were kept as nearly as possible in a horizontal plane; the continuation of the main pipe from the junction will be referred to as "the main branch," the other, called "the branch," makes, in the different experiments, angles varying from 45° to 135° with "the main branch." See Table I for the sizes of the pipes used in each experiment; the different pipes being numbered to avoid errors.

Five sets of experiments were made with $\alpha = 90°$, two sets with $\alpha = 45°$ and two with $\alpha = 135°$, the proper *Y's* and *T's* being used to make good connections at the junction. The angle α is in all cases the angle between the down-stream ends of the branches.

Experiment 1—All pipes were of same size. Diameter, 0''.62.

Experiment 2—Larger pipe reducing to two smaller. Diameters, 0''.858; 0''.62; 0''.62.

Experiment 3—"Main" and "Branch" of the same size; "Main Branch" smaller. Diameters, 0''.858; 0''.84; 0''.62.

Experiments 4, 5, 6, 7, 8, 9—All pipes of same size.

Experiment 4—Diameter, 0''.85, approximately.

Experiment 5—Diameters, 0''.375; approximately.

Experiments 6, 7—Diameters, 0''.61, approximately.

Experiments 8, 9—Diameters, 0''.85, approximately.

Experiments 1 – 5, with $\alpha = 90°$.
Experiments 6 and 8, " $\alpha = 45°$.
Experiments 7 and 9, " $\alpha = 135°$.

All other details of apparatus used are omitted for lack of space. Facilities were available for maintaining a constant head up to about fifty feet.

COMPUTATION AND TABULATIONS OF RESULTS—SYMBOLS AND FORMULÆ.

The following is a list of symbols used throughout our computations and tabulations. The foot, pound, second system is used throughout:

Q = Quantity of water delivered by "Main Branch."

Q' = Quantity of water delivered by "Branch."

v = Velocity; a small figure to the right and below indicates the number of the corresponding pipe.

All the pipes were contracted in diameter at the ends, thus causing a different value for the velocity of efflux.

V = Velocity of efflux from the pipes.

The velocity of efflux was not computed in all cases; where it was not computed, the end diameter of the pipe is given, so that the velocity may be computed if desired. Table I.

h = Head in pressure chamber.

a Refers to the piezometer in main pipe just above the joint, but far enough away so that the readings are not changed by the disturbance at the joint.

b Refers to the piezometer in the "Main Branch" just below the joint.

c Refers to the piezometer in the "Branch" just below the joint.

h_p = Pressure head.

h_v = Velocity head.

H = Algebraic sum of the change of velocity head and change of pressure head, *i. e.*, the total loss of head.

ς Is a coefficient for loss of head. It is computed from the formula:—

$$H = \frac{\varsigma v^2}{2g},$$

$\frac{v^2}{2g}$ being always the velocity head in the main pipe.

H includes the loss of head due to skin friction between piezometers, as well as that due to the disturbance at the junction. The skin fric-

tion, however, was found by computation to be so small that the total loss of head is assumed to be caused by the T or Y.

In obtaining the loss of head due to skin friction between piezometers, the following formula was used :

$$h = 4\, m \frac{l}{d} \frac{v^2}{2\, g}$$

Where m is Fanning's coefficient of skin friction, l equals length, and d the diameter of the pipe. Tables of our observed and computed quantities are to be found at the end of this article. All observed quantities are given because, although not all used by us, they may prove to be interesting data.

In conclusion we would say that we find our results always consistent. A great source of inaccuracy, and one which accounts for variations in the value of s at low velocities, is the difficulty in reading correctly to thousandths of a foot, a mercury manometer graduated to hundredths.

Assuming that there are no errors in computation, the probable cause of the variation in the value of s in any given experiment may be traced directly to the incorrect reading of the differential manometer. All other quantities, which agree among themselves, were by our method of procedure checked, in most cases by repetition of observation.

Some trouble was caused at the higher velocities by the fluctuation of the mercury column, but by taking a mean of the observations a fairly good result was obtained.

We would draw special attention to some of the results shown by the tables. In the first place, the law of variation in amount of flow through the branch, did not appear. With the sizes of pipes used, the variations in (α) between 45° and 135° did not affect the relative amount of flow through the branches.

Again, the relative amount carried by the two branches does not seem to be a function of the head within the limits of our experiments.

With all three pipes of the same size and the two branches of the same length, the "Main Branch" carries the most water, as would be expected. In the cases of the pipes having diameters of 0″.6 and 0″.8, the "Main Branch" carries about 62 % of the total flow ; while in the case of the smallest pipe, the relative amounts are more nearly equal, the "Main Branch" carrying only between 54 and 55 % of the total flow.

With the main pipe large, and both branches smaller, the relative amounts carried by the branches are more nearly equal than in the case where all three pipes are of the same size. This would naturally be expected, the larger pipe acting to a certain degree as a pressure chamber.

We find that the loss of head in passing from the main pipe to the "Branch" is at least double the loss of head in passing from the main to the "Main Branch."

TABLE I.

Tables Showing Arrangement and Sizes of Pipes.

Main Branch.				*Pipe No.*	*Angle*	*Pipe No.*	*Branch.*		
No. of Experiment.	*Length of Pipe.*	*Average Diam.*	*Nozzle end Diam.*		α		*Length.*	*Average Diam.*	*End Diam.*
1	7′ 6½″	.62″	.60″	2	90°	3	7′ 6″	.61″	.60″
2	"	"	"	"	"	"	"	"	"
3	"	"	"	"	"	5	8′ 6½″	.84″	.80″
4	8′ 5¾″	.84″	.8″	6	"	"	"	"	"
5	7′ 4⅜″	.374″	.38″	8	"	9	7′ 4¼″	.373″	.37″
6	7′ 6½″	.62″	.60″	2	45°	3	7′ 6″	.61″	.60″
7	"	"	"	"	135°	"	"	"	"
8	8′ 5¾″	.84″	.80″	6	45°	5	8′ 6½″	.84″	.80″
9	"	"	"	"	135°	"	"	"	"

Main Pipe.

No. of Experiment.	*Length of Pipe.*	*Average Diam.*	*Pipe No.*
1	7′ 6¼″	.61″	1
2	8′ 5″	.858″	4
3	"	"	"
4	"	"	"
5	7′ 6″	.376″	7
6	7′ 6¼″	.61″	1
7	"	"	"
8	8′ 5″	.858″	4
9	"	"	"

Experiment No. 1.

No. of Run.	h Head in Press. Chamb. ft.	Time of Run. m. s.	Q Quant. per. Sec. from M'n Br. Cu. Ft.	Q' from Branch.	Diff. of Total Head. a–b	a–c	Coefficient ς a–b	a–c
1	1.08	24–00	.00346	.00226	.140	0.325	1.215	2.742
2	2.10	18–25	.00534	.00309	.276	.672	1.072	2.611
3	3.10	12–59	.00656	.00392	.401	1.012	1.055	2.662
4	4.00	12–30	.00748	.00451	.539	1.384	1.035	2.657
5	9.27	15–10	.01170	.00712	1.422	3.535	1.108	2.755
6	15.065	12–24	.01530	.00904	2.343	5.810	1.092	2.707
7	20.73	10–15	.01820	.0108	3.285	8.070	1.078	2.649
8	26.70	9–00	.0207	.0122	4.276	10.425	1.090	2.658
9	36.50	11–30	.0240	.0141	5.871	14.570	1.116	2.771
10	44.58	10–00	.0272	.0159	7.260	17.569	1.077	2.606

Experiment No. 2.

No. of Run.	h in ft.	Time of Run. m. s.	Q	Q'	Diff. of Total Head. a–b	a–c	ς a–b	a–c
1	1.10	17–00	.0053	.0042	.092	.213	1.012	2.343
2	2.075	12–00	.0075	.0060	.214	.566	1.214	3.210
3	2.980	10–00	.00916	.00729	.290	.755	1.117	2.984
4	3.990	8–00	.0106	.00858	.409	1.030	1.158	2.915
5	9.59	10–30	.0168	.0136	1.073	2.515	1.213	2.842
6	15.13	8–14	.0216	.0172	1.725	3.995	1.186	2.745
7	20.87	7–2	.0257	.0203	2.365	5.540	1.166	2.732
8	27.85	6–00	.0293	.0231	3.016	7.092	1.142	2.686
9	35.20	7–59	.0334	.0265	4.009	9.286	1.164	2.697
10	40.94	7–43	.0362	.0285	4.876	10.786	1.210	2.676

Experiment No. 3.

No. of Run.	h in ft.	Time of Run. m. s.	Q	Q′	Diff. of Total Head. a–b	a–c	ς a–b	a–c
1	1.015	20–00	.00427	.00673	.102	.181	.882	1.565
2	1.99	13–00	.00624	.00982	.274	.822	1.106	3.318
3	3.03	10–00	.00769	.0123	.389	1.050	1.013	2.735
4	4.035	9–00	.00888	.0140	.501	1.360	1.000	2.715
5	9.45	6–30	.01446	.0226	1.195	3.110	.908	2.362
6	15.13	9–00	.01816	.02849	1.947	4.858	.929	2.319
7	20.86	7–15	.02164	.0338	2.715	6.729	.921	2.282
8	26.60	7–00	.02445	.0384	3.475	8.614	.917	2.272
9	33.70	6–25	.02710	.0424	4.203	10.502	.906	2.262
10	38.86	5–45	.02994	.0470	5.136	12.816	.901	2.250

Experiment No. 4.

No. of Run.	h in ft.	Time of Run. m. s.	Q	Q′	Diff. of Total Head. a–b	a–c	ς a–b	a–c
1	1.005	11–30	.00718	.00488	.034	.201	.386	2.280
2	2.035	7–15	.0112	.00678	.180	.601	.579	1.935
3	3.075	6–0	.0137	.00846	.274	.923	.580	1.956
4	4.025	5–15	.0162	.00972	.419	1.246	.649	1.930
5	9.39	3–15	.0255	.0152	1.026	3.152	.644	1.980
6	15.13	2–30	.0330	.0187	1.665	5.094	.648	1.984
7	20.74	4–30	.0391	.0224	2.350	7.074	.647	1.946
8	26.60	3–45	.0444	.0253	2.971	9.612	.637	2.056
9	33.70	3–30	.0493	.0279	3.517	10.990	.614	1.920
10	37.415	3–15	.0531	.0300	3.979	12.987	.600	1.957

Experiment No. 5.

No. of Run.	*h* *Head in Ft.*	*Time of Run.* *m. s.*	*Q from Main Branch.*	*Q′ from Branch.*	*Diff. Total Head.* *a–b*	*a–c*	*ς* *a–b*	*a–c*
1	1.465	31–00	.000879	.000775	.053	.121	.718	1.640
2	3.160	23–00	.00142	.00127	.115	.264	.594	1.364
3	4.05	35–00	.00165	.00144	.159	.339	.618	1.318
4	8.465	15–15	.00271	.00230	.322	.789	.500	1.225
5	15.13	13–00	.00347	.00300	.542	1.263	.503	1.171
6	20.60	11–00	.00421	.00357	.825	1.885	.507	1.158
7	26.465	12–00	.00482	.00405	.975	2.433	.462	1.152
8	36.765	13–30	.00576	.00481	1.438	3.383	.479	1.126
9	47.135	11–00	.00653	.00546	1.806	4.301	.467	1.112

Experiment No. 6.

No. of Run.	*h* *in ft.*	*Time.* *m. s.*	*Q*	*Q′*	*Diff. Total Head.* *a–b*	*a–c*	*ς* *a–b*	*a–c*
1	2.015	15–30½	.00482	.00295	.362	.580	1.655	2.652
2	4.02	6–30	.01029	.00624	1.093	1.720	1.104	1.737
3	15.32	4–59½	.01440	.00844	3.025	5.105	1.600	2.701
4	29.59	3–56	.02044	.01175	7.426	10.125	1.979	2.697
5	44.00	2–44¾	.02564	.01476	11.292	15.430	1.910	2.609
6	2.99	7–00½	.00682	.00419	.488	.750	1.111	1.709
7	9.39	7–00	.01147	.00673	1.863	3.412	1.552	2.843

EXPERIMENT No. 7.

No. of Run.	*h* in ft.	Time. m. s.	*Q*	*Q'*	Diff. Total Head. a–b	a–c	ς a–b	ς a–c
1	2.00	16–10	.00479	.00296	.309	.560	1.424	2.581
2	4.01	10–00	.00702	.00422	.655	1.145	1.430	2.500
3	9.39	6–10	.0114	.00689	1.743	2.924	1.434	2.405
4	15.32	4–00	.0145	.00855	2.765	4.715	1.438	2.452
5	29.59	3–31	.0206	.01192	5.460	9.422	1.427	2.462
6	44.00	3–10	.0255	.01484	8.362	13.770	1.419	2.336
7	3.00	12–30½	.00594	.00353	.471	.829	1.454	2.559
8	21.45	4–15	.01754	.01031	3.946	6.807	1.404	2.423
9	2.53	12–30	.00558	.00344	.419	0.727	1.425	2.473

EXPERIMENT No. 8.

No. of Run.	*h* Head in Chamber.	Time of Run. m. s.	*Q*	*Q'*	Diff. of Total Head. a–b	a–c	ς a–b	ς a–c
1	2.02	7–45	.01123	.00693	.252	.546	.725	1.571
2	4.03	5–00	.01608	.00999	.544	1.236	.759	1.727
3	15.49	2–35	.03273	.02016	2.125	5.015	.739	1.744
4	24.11	2–00	.04147	.02564	3.621	8.120	.763	1.712
5	35.52	1–48	.05015	.03072	5.302	11.706	.770	1.700
6	9.84	3–15	.02556	.01578	1.363	3.104	.757	1.723

Experiment No. 9.

No. of Run.	h Head in Chamber.	Time of Run.	Q	Q'	Diff. of Total Head.		ζ	
		m. s.			a–b	a–c	a–b	a–c
1	3.97	5–45	.01579	.01008	.542	1.223	.768	1.735
2	15.49	2–44	.03250	.02004	2.294	5.024	.789	1.729
3	24.11	2–14	.04036	.02571	3.712	7.852	.806	1.706
4	35.52	1–40	.04960	.03141	5.347	11.573	.773	1.674
5	9.84	3–38	.02568	.01588	1.376	3.120	.756	1.714
6	2.99	6–00	.01353	.00846	.384	.935	.753	1.834
7	2.00	7–45	.01087	.00696	.277	.596	.829	1.783
8	15.49	2–40	.03241	.02013	2.301	4.921	.792	1.694